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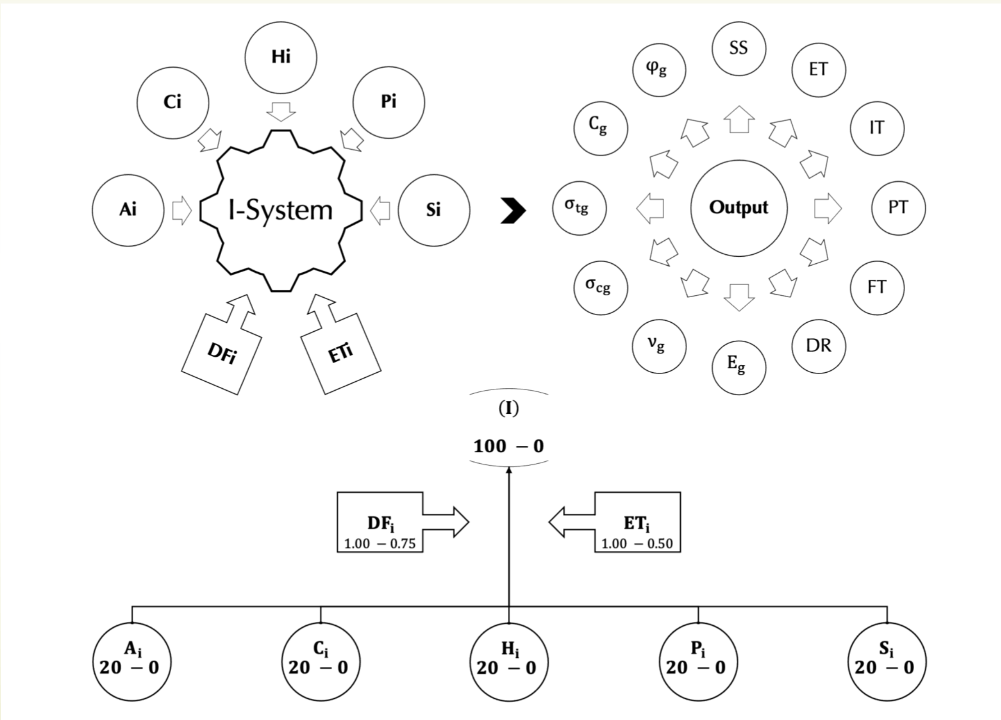

SPECIAL ISSUE : I-SYSTEM OF GROUND CHARACTERIZATION

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Front Cover Photograph: Top Left: Indices and impact factors as input for calculation of I-System; Top Right: (I)-Class and (I)-GC's output as I-System's classification and characterization; Bottom: I-System's rating chart for indices and impact factors.

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(Volume XLVI, No. 1)

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Contents

Editorial

"It may have been in pieces, but I gave you the best of me"

- Jim Morrison

Publication of journal for the society is not a one time job, the process is continues and persistent efforts are required round the year to bring out each issue. Be it collection of technical papers, sending them to our esteemed reviewers, forwarding comments of the reviewers to authors, collecting finalized manuscript from them and finally the acceptance of paper for publication in the journal. During last couple of years, we had tried our best to reduce the publication time. I feel proud to announce that in this issue, we had managed to accept few research papers within one month of initial submission. The credit goes to the elite reviewers of the journal, members of the editorial board and obviously to the authors who managed to submit the corrected manuscripts within the stipulated time line.

This volume carries three very interesting papers by Dr. Bineshian Hoss, Principal, Consultancy Head, Amberg Engineering AG, Australia and who is now an esteemed member of the ISEG editorial board. In order to buttress his association with the society, he had offered to publish the latest 2021 edition of I-System, an engineering classification for ground characterization developed by him after a hard labour and research of about 24 years. Beside, this issue also carries two more articles on I-System, second article highlight the case histories and utility of the I-System software while third one narrates the chronology of its development. Considering the above, it has been decided to dedicate this issue of the Journal of Engineering Geology especially for I-System of Ground Characterization.

It gives me immense pleasure to bring out Vol. XLVI, issue No. 1 for our readers which carries four technical papers on variety of interesting aspects of Engineering Geology and Geotechnics beside three dedicated papers on I-System. The credit for the publication of this Special Issue of the Journal goes to Mr. S.L. Kapil, Secretary, ISEG who has firm belief that innovative works should be eulogised and encouraged. I offer my sincere gratitude to Er. A.K. Singh, President, ISEG also for his enormous encouragement and support to the journal. Last but not the least; I'm deeply indebted to all editorial board members for their cooperation and help without which timely publication of this issue would not have been possible.

September 2021

With regards,

Rahul Khanna
Editor, ISEG

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I-System: Index of Ground-Structure
2021 Edition

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Received December 2020, Accepted May 2021

Abstract

An optimised geotechnical/geomechanical design approach includes empirical, analytical, seismic, and observational stages. Empirical and observational parts of a design are vital in initiation of the approach and in finalisation of judgements for practice and design purposes containing the derivation of ground behaviour, identification of ground hazards, determination of support systems, and characterisation of ground's mechanical properties. Engineering classifications are main part of empirical and observational stages of the design for human made structures in ground; though, they have limitations in application. I-System is a classification as well as a characterisation system for ground that is developed to cross the limitations involved with other classifications. It is comprehensively applicable for civil, mining, and oil & gas structures in ground including but not limited to abutments of bridges and dams, caverns, deep and shallow foundations, embankment and tailing dams, galleries, deep and shallow metro stations, mine stopes, open pits, shafts, slopes, trenches, tunnels, underground spaces and storages, wells, etc. It considers easily derivable geohydrological, geomechanical, geometrical, geophysical, geostructural, geotechnical, and dynamic properties and configuration of ground in relation to the structure together with the method of excavation and construction. It is first published in 2019 based on 22 years' research and verification in design and construction of underground, semi-surface, and surface works in rock and soil; however, since then further developments as well as improvements and clarifications are made. This paper provides the latest edition of I-System (as a full package) and an introduction to I-System Software.

Keywords: (I), (I)-Class, (I)-GC, blast-induced damage, characterisation, classification, Damage Indicator, GCD, GC_{ef}, Ground Conductivity Enhanced Factor, hydraulic conductivity, I-System, Index of Ground-Structure, intact rock, rock mass, soil, SRH, support system, vibration-induced damage, ViD

1. Introduction

Design approach for structures in ground includes 4 important stages as shown in Figure 1 (Bineshian et al, 2019). The design methodology should pass empirical, analytical, seismic, and observational procedures to get the optimised design badge of "good for construction" while empirical and observational parts are playing very crucial role and determinative factors for this purpose. Both parts are quite depended on ground's engineering classification and characterisation.

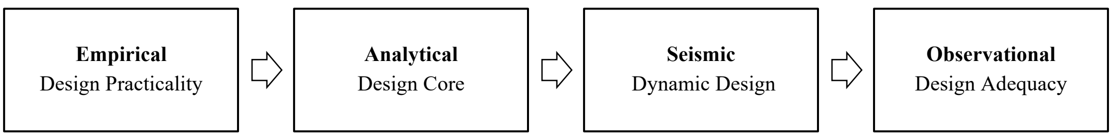
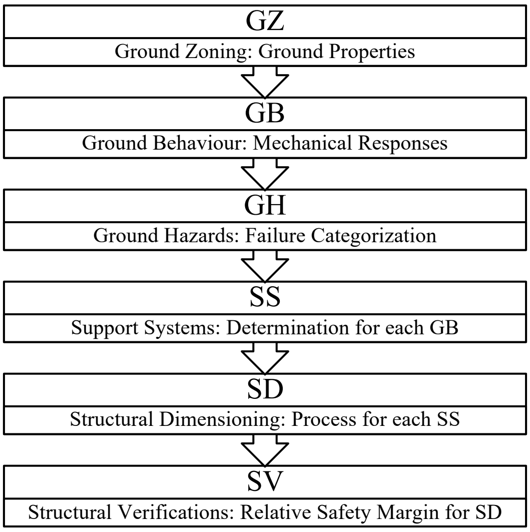
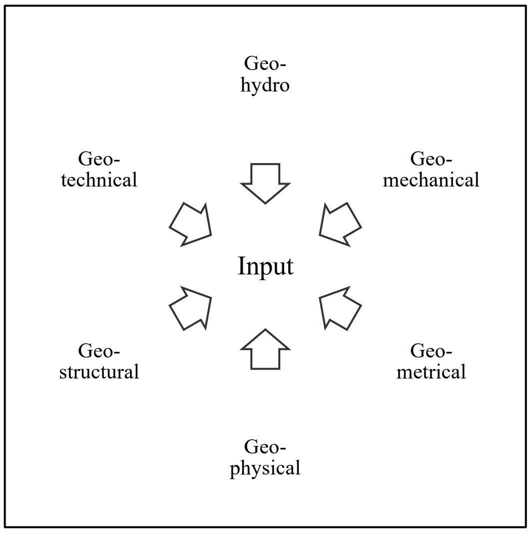


Figure 1. Design approach for structures in ground

Design procedure is presented in Figure 2 based on the design approach explained above. Figure 2a demonstrates a well-defined design procedure and Figure 2b shows the data requirements in a design setting. As a brief definition, Ground Zoning (GZ) is based on ground inherent properties that divides entire length of a tunnel to a group of limited numbers of zones or stretches with similar properties. It eases the identification of ground behaviour and related hazard/s, determination of required support system, procedure of structural dimensioning, and finally verification of the required measures for each zone. GZ is the first stage in design procedure, which is conducted after completion of initial geotechnical/geomechanical investigation in initial phase of study. Empirical classification systems are the important element in identification of GZs.



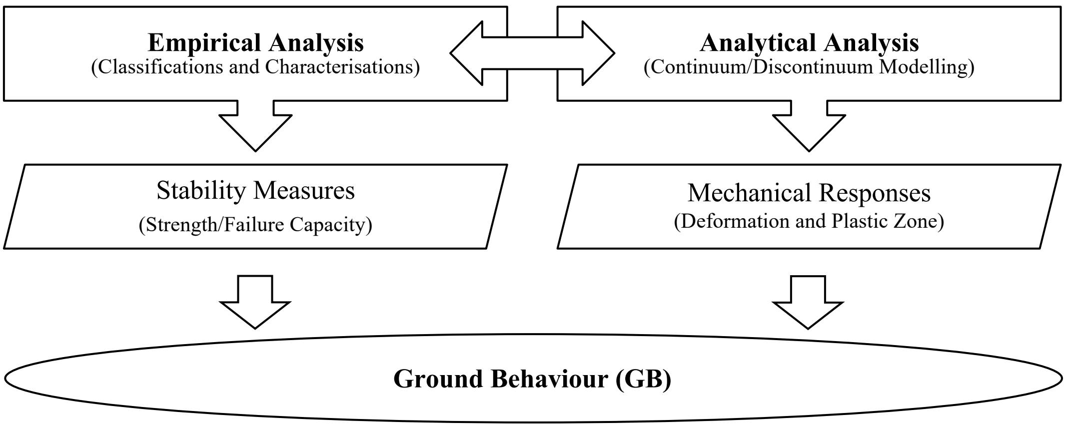
a. Design procedure diagram



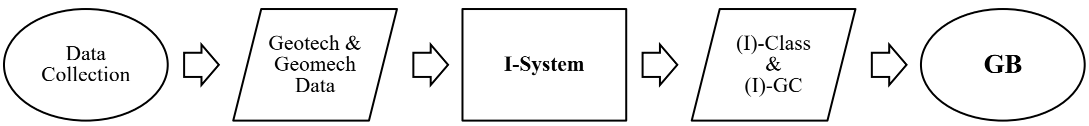
b. Design setting's data requirements

Figure 2. Design procedure for structures in ground

Second and third stages in a healthy design procedure are identification of the Ground Behaviour (GB) and associated Ground Hazard/s (GH) respectively. Russo and Grasso (2007) proposed an approach to identify excavation behaviour based on continuum equivalent and equilibrium models; however, in this paper, it is produced for continuum and discontinuum media by combined analytical and empirical modelling as principal concept in identification of GB (Figure 3a). As can be seen in Figure 3a, classification systems are used in empirical analysis for identification of GB. Figure 3b represents the same by a fully empirical approach using I-System (Figure 3b) as a classification and characterisation system (Bineshian, 2019a, 2019b).



a. Analytical/empirical stress analysis approach in identification of GB



b. I-System approach in determination of GB (Bineshian, 2019a, 2019b)

Figure 3. Flowcharts representing two ways to identify the GB; stress analysis and I-System

Figure 4 represents most expected Ground Hazards (GH) from the identified GB that should be considered within the design procedure (Figure 2a).

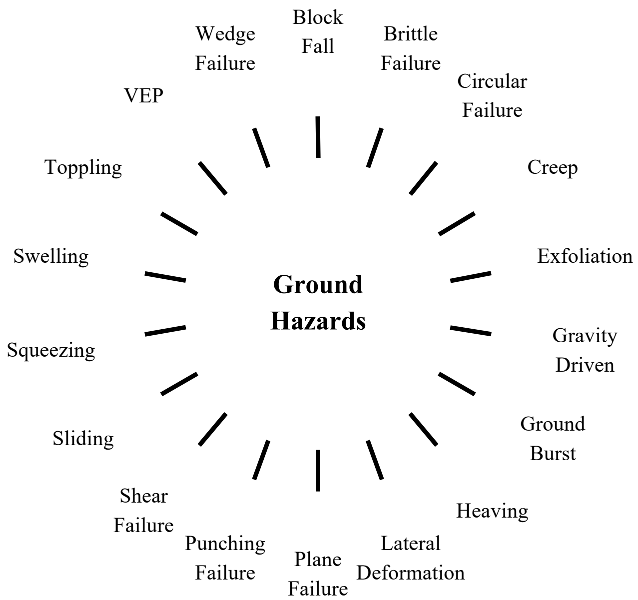


Figure 4. Ground Hazards (GH) expected from the identified GB

In the fourth stage of a design procedure (Figure 2a), Support System/s (SS) should be selected from choices of solutions required for each GZ and related GB and GH. Again, the need for a comprehensive and suitable classification system is recognised to be vital to find the best solution/s for each mechanical response and associated hazard/s.

Further to selection of suitable solution/s as SS for each GH, the measures (either primary or final SS) should be dimensioned (calculation part of design approach; fifth stage in Figure 2a) and verified (defining the relative safety margins; last stage of design approach in Figure 2a). Probabilistic Convergence-Confinement method (e.g., Carranza-Torres, 2004) can be used for Structural Dimensioning (SD). In Structural Verification (SV), Limit State Design (LSD) that known as Load and Resistance Factor Design (LRFD) method is used (McCormac, 2008). LSD itself has two procedures in design verification; Ultimate Limit State (ULS) and Serviceability Limit State (SLS). ULS includes checking against generated bending moment, axial forces, and shear forces (EN 1990:2002 E). On the other hand, SLS checks the generated crack width in the structure (e.g., crack width < 0.30 mm as per IS 456:2000). Figure 5 illustrates the SV procedure as the last stage in a design procedure (Figure 2a) required for plain or reinforced concrete structure (for primary or final SS).

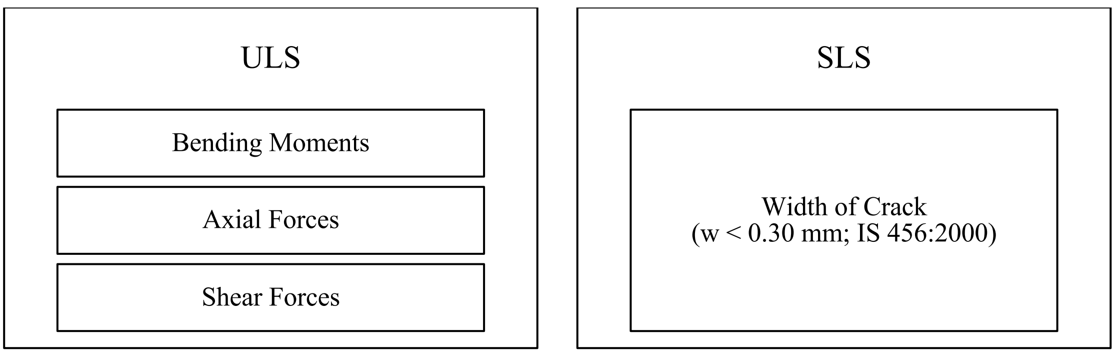


Figure 5. Structural Verification (SV) check

As per design procedure and requirement explained here, it is proved that engineering classifications are the main part of the empirical and observational design elements in a healthy design approach and design procedure shown in Figures 1 and 2 (Bineshian, 2012, Bineshian and Ghazvinian, 2012a and 2012b). Comprehensiveness and practicality of the engineering classifications are essential to make them appropriately applicable in NATM, NMT, SEM, SCL, etc.; however, existing engineering classifications come with limitations in use for both rock and soil.

Limitations, inaccuracy, and imprecision involved with existing classifications make engineers uncertain in determination and dimensioning of structures specially when they encounter ground complications (Bineshian, 2014, Bineshian, 2017, Bineshian et al, 2019). RMR and Q are popular existing classifications developed by Bieniawski (1973) and Barton et al (1974) respectively. They are only applicable for rock medium. RMR is proposed for surface and underground works but its water pressure consideration is doubtful, quantification of joint orientation is uncertain, and the effect of water on rock mass is inattentive (Bineshian et al, 2013). Q is proposed for tunnels merely, which comes with several limits in input parameters including discontinuity's aperture, orientation, persistency, size, and rock strength. Palmstrom and Broch (2006) stated that there is a shortcoming in most existing classifications when observed rock mass characteristics are used to estimate the conditions for design without including input of the excavation method. An excavation damage factor or similar should be applied, but none of the existing empirical or other tools in rock engineering makes use of this (Palmstrom and Broch, 2006).

I-System is developed to be used as a comprehensive classification and characterisation system for ground (Bineshian, 2019b). It is verified against varieties of ground and scrutinised in several projects through 22 years research to address and resolve the aforesaid issues involved with existing classifications (Table 1). I-System provides prediction of ground behaviour together with recommendations on required Support System/s (SS), Excavation Technique/s (ET), Instrumentation Technique/s (IT), Prevention Technique/s (PT), and Forecast Technique/s (FT) followed by Design Remark/s (DR) as well as estimation for important mechanical properties of ground. Its output is optimised by analytical, numerical, and observational methods to compensate the demerits of existing classifications and strengthen its comprehensiveness.

Table 1. Application summary for popular existing engineering classifications compared to I-System

Applications / System	Media		Structure (Civil, Mining, Oil and Gas)	
	Rock	Soil	Surface	Underground
RMR (Bieniawski, 1973)	a	n/a	c/a	a
Q (Barton et al, 1974)	a	n/a	n/a	a
I-System (Bineshian, 2019b)	a	a	a	a

a Applicable
c/a Conditionally Applicable
n/a Not Applicable

This paper is a 2021 edition of I-System in a full package, which is further developed by providing vibration-induced damage (ViD) assessment methods, pull length advisor, and systematic bolting calculator. It provides further illustrations, details, clarifications, and updates to I-System as well as introducing I-System Software as a design utility that eases the use of I-System while expected accuracy is obtained in calculation.

2. I-System: Definition

Providing a solution to engineers in their challenges with complicated ground conditions is the key perception and approach in development of this all-in-one classification and characterisation system for ground in accord with real condition to deliver design parameters and practical recommendation/s. Also, it has been in mind to provide a trusted utility for empirical part of design. In development of this system, drawbacks and limitations of other classifications (e.g., RMR and Q) are properly addressed and consequently resolved (Bineshian, 2019a, 2019b). This comprehensive classification and characterisation system for ground (rock and soil) entitled “Index of Ground-Structure” or in short form “I-System”. It is conceptually different from any existing classifications due to its applicability for varieties of ground conditions and structures and its comprehensiveness in providing accurate and precise prediction of ground behaviour based on several geomechanical hazards (failure mechanisms) studied in course of development. Its range of application (Figure 6) in design and/or practice includes underground structures (caverns, deep or underground metro stations, exploration and grouting galleries, mine stopes, shafts, tunnels of any type or method, underground spaces, underground storages, wells, etc.), semi-surface structures (bridge abutments, dam abutments, deep foundations, shallow metro stations including open-cut and cut & cover, etc.), and surface structures (embankment dams, open pits, shallow foundations, slopes, tailing dams, trenches, etc.).

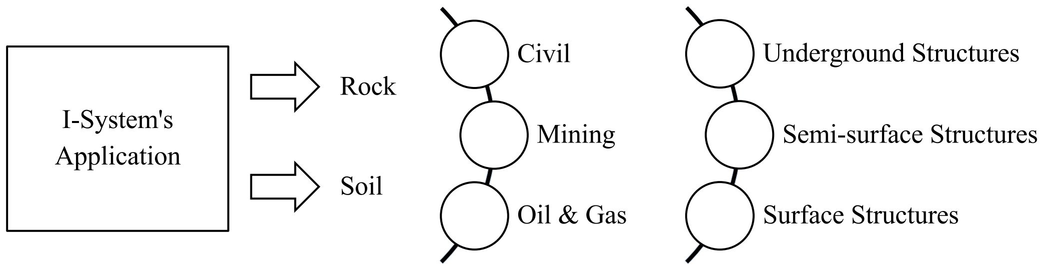
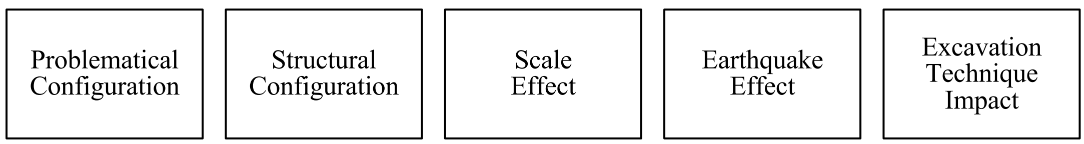


Figure 6. Range of application of I-System

It is the first ever classification, which is applicable for both rock and soil that considers ground’s problematical and structural configurations, opening’s scale effect, earthquake’s negative effect, and excavation technique’s impact (Figure 7a). Besides, it is the first ever classification that carefully provides prediction for special ground behaviour including but not limited to Squeezing, Swelling, and Heaving (SSH), Time Dependent (TD), Visco-elasto Plastic (VP), fully plastic, gravity driven (GD), and Burst Prone (BP) condition (Figure 7b).



a. First ever features covered by a classification system

b. Special ground behaviour covered by I-System
Figure 7. Most important features covered by I-System

I-System is verified in a wide varieties of challenging ground conditions to ensure that a suitable estimation is obtained in classification and characterisation. It provides recommendations on determination of primary and final SS, required ET for encountered condition, proper IT for monitoring, appropriate PT against possible failures, verified FT to predict the ground condition ahead, and practical DR that is helpful in understanding of ground behaviour, failure mechanism, and load configuration (Section 4). Moreover, it characterises the ground by deriving the mechanical properties (Section 5) that can be used as input for SD in design procedure.

It is intended that I-System to have key indices to enable an appropriate modelling of ground-structure behaviour to the full (Figure 8). It includes five indices to define the mechanical response of ground in relation to the structure. Furthermore, it has two impact factors to define the impact of Dynamic Forces (DF_i) and Excavation Technique (ET_i) on structure. Indices and impact factors in I-System (Figure 8) are based on easily derivable main properties (i.e., key geomechanical, geostructural, geohydrological, geotechnical, geophysical, and geometrical features; Figure 2b) and determinant seismic and excavation factors that affecting the ground-structure response (Figure 7a).

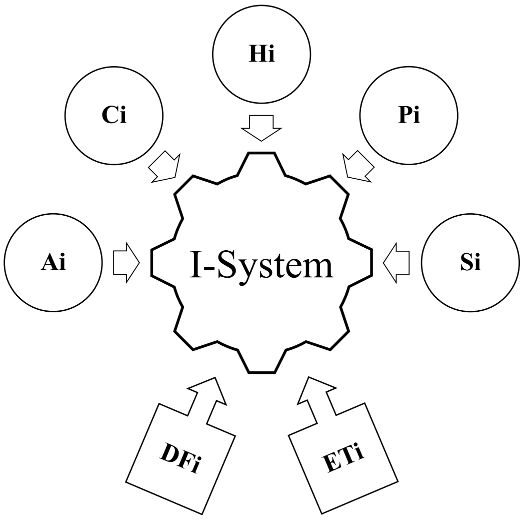


Figure 8. I-System calculation; indices and impact factors

Eq 1 represents I-System in a mathematical form entitled "(I)". Eq 2 to 8 defines the indices and the impact factors for (I) as follows:

$$(\mathrm{I}) = (\mathrm{A_i} + \mathrm{C_i} + \mathrm{H_i} + \mathrm{P_i} + \mathrm{S_i}) \times \mathrm{DF_i} \times \mathrm{ET_i} \quad (1)$$

$$\mathrm{A_i} = (\mathrm{a_{dn}} + \mathrm{a_{ds}} + \mathrm{a_{di}}) \times \mathrm{a_{da}} \times \mathrm{a_{dd}} \times \mathrm{a_{df}} \times \mathrm{a_{dp}} \quad (2)$$

$$\mathrm{C_i} = \mathrm{c_{pc}} \times \mathrm{c_{sc}} \quad (3)$$

$$\mathrm{H_i} = \mathrm{h_{gc}} \times \mathrm{h_{gs}} \quad (4)$$

$$\mathrm{P_i} = [\mathrm{p_{cc}} + \mathrm{p_{dc}} + (\mathrm{p_{ps}} \times \mathrm{p_{pm}})] \times \mathrm{p_{bw}} \;\&\; \mathrm{p_{bw}} = f(\mathrm{V_p}, \mathrm{V_s}) \quad (5)$$

$$\mathrm{S_i} = \mathrm{s_{cs}} \times \mathrm{s_{se}} \quad (6)$$

$$\mathrm{DF_i} = f(\mathrm{PGA_{SD}}, \mathrm{ERZ}, \mathrm{MSK}) \;\&\; \mathrm{PGA_{SD}} = f(\mathrm{PGA}, \mathrm{SF}, \mathrm{MSF}) \quad (7)$$

$$\mathrm{ET_i} = f(\mathrm{ET}, \mathrm{PPV}) \quad (8)$$

where;
(I) I-System's value
A_i Armature Index
C_i Configuration Index
H_i Hydro Index
P_i Properties Index
S_i Strength Index
DF_i Dynamic Forces Impact
ET_i Excavation Technique Impact

I-System's value ranges between 100 – 0 and classifies the ground-structure interaction to 10 classes as (I)-01 to (I)-10 from best to worst class. The indices of A_i, C_i, H_i, P_i, and S_i have 20 per cent share out of a total score of 100. DF_i and ET_i are factors ranging between 1 – 0.75 and 1 – 0.50 respectively, which impact the summation of indices (Figure 9). Indices are defined in the Section 3. Full definition of the parameters is available in Section 10.

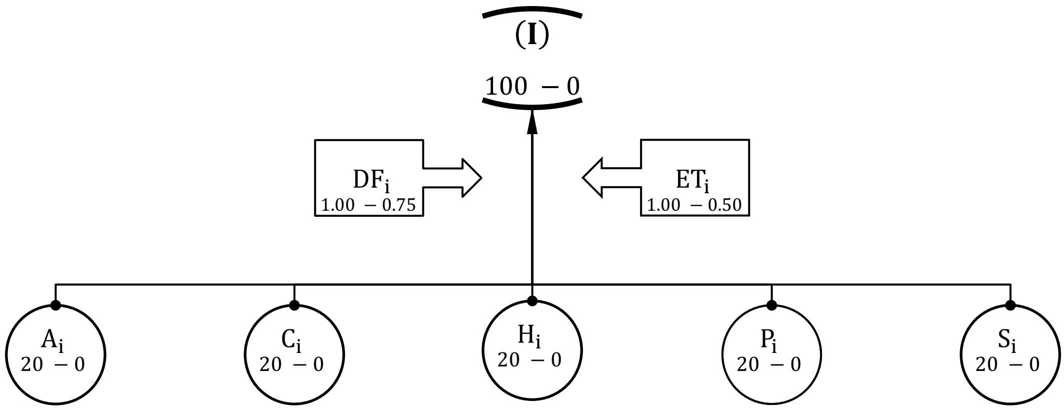


Figure 9. I-System's scoring diagram

I-System is applicable for estimation of quality of ground in relation to the structure at any scale and type. It assists with empirical and observational parts of the design approach (Figure 1). I-System is applicable in design procedure and/or in practice (Figures 2 and 3) for:

- categorizing the ground properties in relation to Ground Zoning (GZ),
- discovering Ground Behaviour (GB),
- identifying associated failure mechanism/s (Figure 4) as Ground Hazard/s (GH),
- determining the required Support System/s (SS); Section 4, and
- assisting in Structural Dimensioning and Verification (SD and SV) by characterizing the most important mechanical properties of ground (Section 5).

It is also applicable to (Tables 9 - 12 in Section 4):

- find the appropriate technique/s for excavation further to the determination of the required support system/s (ET),
- select suitable option for instrumentation/monitoring during construction (IT),
- implement the proper technique for prevention of hazard/s (PT), and
- designate the required technique for forecasting/prediction (FT).

I-System is developed to serve the above-stated purposes for underground, semi-surface, and surface structures in the field of civil, mining, and oil and gas.

3. Indices and Impact Factors

I-System (Eq 1) includes 5 indices and 2 impact factors (Figures 8 and 9) with mathematical form of Eq 2 to 8. In this section all associated parameters of each index are defined in details. Derivation of parameters from ground and their use in I-System is confusion-free; consequently, selection of the input data is certain, which makes the classification's output accurate and credible. Section 10 provides a complete list of definitions for abbreviations, parameters, and short forms used in this paper.

3.1. Armature Index

A_i is the Armature Index (Eq 1 and 2) as ground's skeleton armature, which is intended to model the most important geomechanical aspects of rock mass as a ground medium through the discontinuity properties of ground. A_i has 20 score out of 100 (Figure 9). Table 2 defines parameters of A_i.

Table 2. Armature Index (A_i): a_{dn}, a_{ds}, a_{di}, a_{da}, a_{dd}, a_{df}, a_{dp}

Discontinuity Number/s	a_{dn}	Discontinuity Set/s	a_{ds}	Discontinuity Inclination	a_{di}
0 - 9	10.00	0	10.00	[IF ($a_{dn} \geq 2.50$ & $a_{ds} \geq 4.00$) THEN↓ ELSE 0]	
10 - 14	7.50	1	9.00	n/a or Granular	0.00
15 - 19	5.00	2	7.00	0 - 10	-1.00
20 - 24	2.50	3	4.00	11 - 30	-1.50
≥ 25	0.00	≥ 4	0.00	31 - 60	-2.00
n/a or Granular	0.00	n/a or Granular	0.00	61 - 90	-2.50

Discontinuity Aperture	a_{da}	Discontinuity Disintegration	a_{dd}	Discontinuity Friction	a_{df}	Discontinuity Persistency	a_{dp}
n/a or Granular	1.00	n/a or Granular	1.00	n/a or Granular	1.00	n/a or Granular	1.00
Tight	1.00	Unweathered/Unaltered	1.00	High Friction - Rough/Uneven	1.00	< 0.90 × D*	1.00
Semi-Tight	0.95	Semi-Integrated	0.95	Moderate Friction - Nonsmooth	0.95	≥ 0.90 × D*	0.90
Open	0.90	Weathered/Altered	0.90	Low Friction - Smooth/Even	0.90		

* For semi-surface and surface structure, "D" should be replaced with "B", which is the Berm's width in a slope or in a trench
a_{da} Factor related to "Discontinuity Aperture" that is based on the most unfavourable opening of the discontinuities
a_{dd} Factor related to "Discontinuity Disintegration" that is based on the worst weathering or alteration of surface of the discontinuity sets
a_{df} Factor related to "Discontinuity Friction" that is based on the least friction condition of discontinuity sets
a_{di} Score related to "Discontinuity Inclination" that is based on dip angle of the most unfavourable discontinuity set
a_{dn} Score related to "Discontinuity Number/s" that is based on number of individual discontinuities per meter of a horizontal or vertical scanline or average of number of discontinuities per meter of horizontal and vertical scanline
a_{dp} Factor related to "Discontinuity Persistency" that is based on the most unfavourable discontinuity set
a_{ds} Score related to "Discontinuity Set/s" reflecting the number of sets of discontinuities
D Diameter, width, or height (mm) of underground opening (the greater value)
Granular A definition describing the soil; a medium, which is not considered as discontinuum
n/a Not Applicable

It should be noted that, if "a_{dn}" and "a_{ds}" are zero, the score for "a_{di}" to be assigned as zero; it happens when the number of discontinuities is ≥ 25 and number of discontinuity sets is ≥ 4. It means that the inclination for the most unfavourable or critical discontinuity set is not easily derivable. In this case, the medium tends to be homogeneous and isotropic due to generated uniform texture – by presence of high number of discontinuities as well as discontinuity sets – that is subject to mechanical response related to continuum mechanics' principles.

Moreover, if the medium is soil mass, "n/a or Granular" to be selected for each parameter from Table 2; otherwise, for rock (intact or mass) the suitable parameter other than "n/a or Granular" to be selected.

3.2. Configuration Index

C_i is the Configuration Index (Eq 1 and 3) as ground's problematical and structural configuration that contains important problematical geostructural features of rock and/or soil. C_i has 20 score out of 100 (Figure 9). Table 3 defines parameters of C_i.

Table 3. Configuration Index (C_i): c_{pc}, c_{sc}

Problematical Configuration of Ground	c_{pc}
Homogeneous or Isotropic or Jointless or Granular*	1.00
Fractured - Slightly	0.95
Faulted - Brittle Single	0.90
Folded - Anticline/Syncline	0.85
Folded - Dome/Basin	0.80
Fractured - Moderately	0.75
Faulted - Graben/Horst	0.70
Folded - Complex/Plunging	0.65
Fractured - Highly	0.60
Faulted - Brittle/Ductile Multiple	0.55
Differed - Unconformities	0.50
BP - High Stress Zone; High Overburden - e.g., Rock Burst, Coal Burst	0.45
Tectonised - Complex of Geostructures	0.40
Sheared - High Shear Stresses - e.g., Mylonite	0.35
TD - Flaky/Micaceous/Cleated - Coals, Mudstone, Phyllite, Schist, Shale, Slate, Young Sandstones	0.30
VP - Incremental-Sudden Large Shear Movement, Cyclic Mobility-Flow Liquefaction, Limited-Continuous Debris Discharge - Flowing/Overrunning	0.25
Structural Configuration of Ground	c_{sc}
Continuum Massive Rock**	20.00
Layered Rock (> 100 cm)	17.00
Layered Rock (100 - 10 cm)	15.00
Clastic Breccia/Conglomerate	13.00
Layered Rock (< 10 cm)	11.00
Foliated/Laminar/Platy Rock	9.00
Coarse Grained Skeleton Soil	7.00
Cohesive Matrix Skeleton Soil	4.00
Single Grained Skeleton Soil - Dense Texture	2.00
Single Grained Skeleton Soil - Loose Texture	0.00

* "Homogeneous or Isotropic or Jointless or Granular" represents a ground condition that it is homogenous and/or isotropic, which is jointless like intact rock or granular like soil mass. Abstractly, this option to be selected when the ground is intact rock or soil mass.
** "Continuum Massive Rock" represents a ground, which is massive medium rather than layered one; e.g., intact rock or unlayered and structurally interlocked rock mass.
c_{pc} Impacting factor related to "Problematical Configuration" of ground indicating ground's tectonic state
c_{sc} Score of "Structural Configuration" of ground (an effect of ground's texture, fabric, and structure)
BP Burst Prone - ground condition with rock burst or coal burst behaviour
TD Time Dependent - ground condition with time dependent shearing behaviour such as squeezing/swelling/heaving behaviour, or even creep
VP Visco-elasto-Plastic - ground condition as visco-elasto-plastic to fully plastic behaviour that contains elastic component/s together with viscous component/s, which makes ground strain rate time dependence; however, due to losing energy during static/dynamic loading cycle, its behaviour converts to fully plastic and may flows like a viscous substance.

In selection of right description for "Problematical Configuration" in Table 3, if the medium is jointless like intact rock or if it is granular like soil mass, "Homogeneous or Isotropic or Jointless or Granular" to be picked. Furthermore, to select "Structural Configuration" correctly, if ground contains unlayered and structurally interlocked rock mass rather than layered one or it contains intact rock, "Continuum Massive Rock" to be picked.

3.3. Hydro Index

H_i is the Hydro Index (Eq 1 and 4) as hydro effect on ground's mechanical behaviour and its hydro related properties. It is a function of GCD (Ground Conductivity Designation; Appendix 1, Bineshian, 2020a) or Wetness diagram (Figure 10) and softness due to presence of water (in scale of Mohs). H_i has 20 score out of 100 (Figure 9). Table 4 defines parameters of H_i.

Table 4. Hydro Index (H_i): h_{gc}, h_{gs}

Ground Conductivity (GCD) or [Wetness]	h_{gc}	**Ground Softness** (Mohs)	h_{gs}
(≤ 0.99) or [Dry]	20.00	≥ 7	1.00
(1 - 1.99) or [Humid]	19.00	6	0.60
(2 - 2.99) or [Damp]	18.00	5	0.50
(3 - 4.99) or [Moist]	16.00	4	0.40
(5 - 6.99) or [Leak]	15.00	3	0.30
(7 - 9.99) or [Wet]	13.00	2	0.20
(10 - 14) or [Drip]	11.00	1	0.10
(15 - 24) or [Shower]	9.00	Moulded by Light Finger Pressure	0.05
(25 - 49) or [Flow]	6.00	Exuded between Fingers	0.00
(50 -99) or [Gush]	3.00		
(≥ 100) or [Burst]	0.00		

GCD Ground Conductivity Designation (Bineshian, 2020a; Appendix 1) as a criterion to score the hydraulic conductivity of ground; it is listed in the table inside parentheses – ( ); it is not mandatory to use GCD value to derive correct value for gc from Table 4; instead, Wetness diagram (Figure 10) can be considered for the same in conjunction with Table 4.

h_{gc} Score assigned to "Ground Conductivity" that is measured using GCD or selected from Wetness diagram as criterion for hydropressure effect on ground

h_{gs} Impact factor related to "Ground Softness" that is considered as an effect of water on medium or infilling material (Mohs)

Wetness A diagram defined here to categorise the ground's water content, which is classifying the ground water condition (observational identification) in 11 ranges (Figure 10); it is listed in the table inside brackets – [ ]

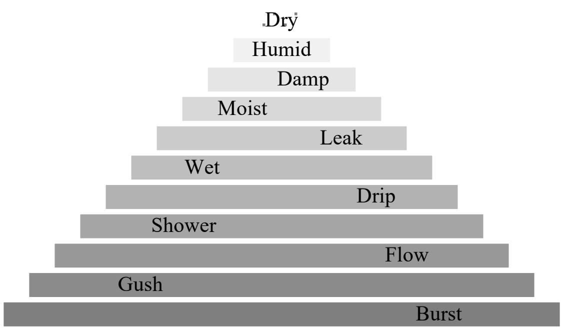


Figure 10. Wetness diagram

GCD provides a quantitative measure for "Ground Conductivity". If GCD test is not used then observational ground water condition to be considered as a criterion for scoring the "h_{gc}" using the Wetness diagram (Figure 10) in conjunction with Table 4. GCD is listed in Table 4 inside the parenthesis "( )".

Wetness diagram provides a qualitative description for "Ground Conductivity" based on observational identification for ground's hydraulic conductivity. It classifies the ground wetness into 11 ranges from dry to water burst. Wetness diagram is listed in Table 4 inside the brackets "[ ]". It is the choice of designer, engineer, or geologist to use GCD or Wetness diagram as per site condition.

3.4. Properties Index

P_i is the Properties Index (Eq 1 and 5) as ground shear properties by way of a definition as a function of texture, fabric, shape, and size of soil materials together with body wave velocity. P_i is considered to be an important part of I-System to model essential geotechnical characteristics of ground as part of the comprehensiveness of the system in applicability for varieties of ground, which in this index, it is the soil medium. P_i has 20 score out of 100 (Figure 9). Table 5 defines parameters of P_i.

Table 5. Property Index (P_i): p_{cc}, p_{dc}, p_{ps}, p_{pm}, p_{bw}

Cohesiveness Consistency	p_{cc}	**Denseness Consistency**	p_{dc}
Indurated	8.00	Never Indented by Thumbnail	6.00
Large Size Particles	6.50	Indented Hardly by Thumbnail	5.00
Picked Difficult	5.00	Indented by Thumbnail	4.00
Picked Easily	3.50	Indented by Thumb	3.00
Shovelled Difficult	2.00	Moulded by Strong Finger Pressure	2.00
Shovelled Easily	0.50	Moulded by Light Finger Pressure	1.00
Foot Imprint Easily	0.00	Exuded between Fingers when Squeezed in Hand	0.00

Particles' Size	p_{ps}	**Particles' Morphology**	p_{pm}	**Body Wave Velocity** m/sec (Vp) or [Vs]	p_{bw}
n/a e.g., Rock	3.00	n/a e.g., Rock	2.00	(≥ 6000) or [≥ 3300]	1.00
Boulder	3.00	Angular	2.00	(5999 - 5000) or [3299 - 2900]	0.90
Cobble	2.50	Sub-angular	1.50	(4999 - 4500) or [2899 - 2600]	0.80
Pebble	2.00	Flat	0.75	(4499 - 4000) or [2599 - 2200]	0.70
Gravel	1.50	Rounded	0.00	(3999 - 3500) or [2199 - 2000]	0.65
Sand	1.00			(3499 - 3000) or [1999 - 1500]	0.60
Silt	0.50			(2999 - 2500) or [1499 - 1000]	0.55
Clay	0.00			(2499 - 2000) or [999 - 750]	0.50
				(1999 - 1000) or [749 - 300]	0.45
				(≤ 999) or [≤ 299]	0.40

n/a	Not Applicable; it should be chosen when the ground is rock including intact rock or rock mass
p_{bw}	Factor related to "Body Wave Velocity" including Vp or Vs as geophysical properties of ground that corrects P_i; Body Wave Velocity is derived either from reliable references (considering the type of materials of ground) or is measured using geophysical methods
p_{cc}	Score related to "Cohesiveness Consistency" that is an important shear property of soil (cohesion)
p_{dc}	Score related to "Denseness Consistency" that is an important shear property of soil (non-cohesiveness; friction)
p_{pm}	Influencing parameter related to "Particles' Morphology" that is a function of shape of soil's grains/granules
p_{ps}	Influencing parameter related to "Particles' Size" that is a function of size of soil's grains/granules"
Rock	Intact rock or rock mass
Vp	Primary Wave Velocity (m/sec); it is listed in the table inside parentheses – ( )
Vs	Shear or Secondary Wave Velocity (m/sec); it is listed in the table inside brackets – [ ]

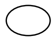
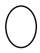
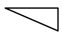
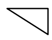
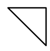
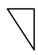
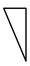
As it is stated in the footnote of Table 5, "Body Wave Velocity" can be derived from reliable references or it can be measured using geophysical surveying method/s. It is recommended to use the geophysical technique/s to derive Vp and/or Vs; however, it is not compulsory to measure "Body Wave Velocity" by conducting geophysical surveys when conduction of measurement is not feasible or practicable. Besides, it should be noted that either Vp or Vs can be used in selection of proper value for "P_{bw}" in Table 5.

Furthermore, to clarify the term "Rock" in Table 5, it should be selected if the ground is intact rock or rock mass, but if the medium contains conglomerate or breccia with poor matrix that stone pieces are easily detached from the matrix, options other than "n/a" and "Rock" to be chosen.

3.5. Strength Index

S_i is the Strength Index (Eq 1 and 6) representing ground's strength behaviour under confining condition. Due to importance of this index in I-System, key parameters of both ground and structure are considered to define this index. In definition of S_i, unconfined compressive strength of ground, scale effect, shape factor of the structure, and stress ratio between vertical and horizontal virgin stresses at the location or depth of placement of structure is considered. S_i has 20 score out of 100 (Figure 9). Table 6 defines parameters of S_i.

Table 6. Strength Index (S_i): s_{cs}, s_{se}

Compressive Strength (UCS)	s_{cs}	**Scale Effect**	**Shape**	s_{se}	
≥ 200 MPa	20.00	UndS - B/H		$\sigma_v \geq \sigma_h$	$\sigma_v < \sigma_h$
199 - 150 MPa	19.00	≥ 2.50		0.80	1.00
149 - 100 MPa	18.00				
99 - 75 MPa	16.00	= 1.90 - 1.30		0.85	0.95
74 - 50 MPa	14.00				
49 - 30 MPa	12.00	= 1.20 - 0.80		0.90	0.90
29 - 20 MPa	10.00				
19 - 10 MPa	9.00	= 0.70 - 0.50		0.95	0.85
9 - 5 MPa	8.00				
4.90 - 2 MPa	7.00	≤ 0.40		1.00	0.80
1.90 - 1 MPa	6.00				
999 - 400 KPa	5.00	SurS - B/H		s_{se}	
399 - 200 KPa	4.00	≥ 2.50		1.00	
199 - 100 KPa	3.00				
99 - 50 KPa	2.00	= 1.90 - 1.30		0.95	
49 - 30 KPa	1.00				
≤ 29 KPa	0.00	= 1.20 - 0.80		0.90	
		= 0.70 - 0.50		0.85	
		≤ 0.40		0.80	

B/H Underground, semi-surface, or surface structures' shape or scale factor as ratio of horizontal span to height of underground opening or ratio of width of berm to height of slope or trench
s_{cs} Score related to "Compressive Strength" as Unconfined Compressive Strength (UCS) of ground
s_{se} "Scale Effect" factor
SurS Surface or Semi-surface Structure
UCS Unconfined Compressive Strength
UndS Underground Structure
σ_h Horizontal Stresses at the location or at the depth of the placement of the structure
σ_v Vertical Stresses at the location or at the depth of the placement of the structure

In Table 6 a wide range of strength from below 29 KPa to over 200 MPa is considered to cover varieties of very weak soil to very strong rock. Higher range of strength is given in MPa while ranges below 1 MPa is given in KPa that makes the strength values more expressive.

Derivation of "s_{se}" for underground structure from Table 6 requires two steps:

1. Select matching shape or "Scale Effect" range based on B/H.
2. Pick the proper "s_{se}" from either $\sigma_v \geq \sigma_h$ column or $\sigma_v < \sigma_h$ column.

Derivation of "s_{se}" for surface or semi-surface structure from Table 6 is as follows:

1. Select the proper range for "Scale Effect" based on B/H.
2. Pick the proper "s_{se}" from the associated column.

3.6. Dynamic Forces Impact

DF_i is the Dynamic Forces Impact (Eq 1 and 7) on the ground-structure behaviour that represents effect of earthquake. Table 7 defines values of DF_i as a function of Scaled Design Peak Ground Acceleration (PGA_{SD}), Earthquake Risk Zone (ERZ), or Medvedev-Sponheuer-Karnik (MSK) Scale (Medvedev and Sponheuer, 1969). If PGA_{SD} is selected to be used for derivation of DF_i, it should be scaled by designer (Eq 9) that may require the ground motion time history data to produce the time-acceleration curve; consequently, scaling factor (SF) to be calculated using the PGA derived from the curve and the desired PGA; accordingly, the time-acceleration plot is scaled. This is a simple procedure that designers who performs dynamic response spectrum analysis are familiar with. Magnitude Scaling Factor (MSF) is another way for scaling the desired PGA; Eq 10 (Idriss, 1999) is an example that is derived for cohesionless soils; however, similar relationships (Idriss and Boulanger, 2008, 2010, Boulanger and Idriss, 2014) for cohesionless soils or any other reliable MSF relationships for cohesive soils may be used for derivation of MSF. When PGA_{SD} is produced, Table 7 to be used to pick the associated value of DF_i; otherwise, if use of ERZ or MSK is preferred, subsequently the earthquake zoning map for project area from reliable references to be used for determination of ERZ or MSK and then related DF_i to be picked from Table 7. ERZ is categorised in 7 classes of damage risk zones as shown in Table 7; EH (MSK XI-XII), VH (MSK IX-X), H (MSK VII-VIII), M (MSK V-VI), L (MSK IV), VL (MSK III), and EL (MSK I-II). DF_i ranges between 1.00 to 0.75 (Figure 9).

$$SF = PGA_{SD} \div PGA \ \& \ PGA_{SD} = SF \times PGA \tag{9}$$

$$MSF = 6.9 \times e^{\left(\frac{-M}{4}\right)} - 0.058 \leq 1.8 \ \& \ PGA_{SD} = MSF \times PGA \tag{10}$$

where;
M Moment Magnitude of Earthquake
MSF Magnitude Scaling Factor
PGA Peak Ground Acceleration (g); maximum ground acceleration during earthquake
PGA_{SD} Scaled Design Peak Ground Acceleration (g); scaled desired PGA
SF Scaling Factor

Table 7. Dynamic Forces Impact (DF_i)

(PGA_{SD}) or [ERZ] or {MSK}	**DFi**
(< 0.05g) or [EL] or {I-II}	1.00
(0.06g - 0.10g) or [VL] or {III}	0.99
(0.11g - 0.15g) or [L] or {IV}	0.97
(0.16g - 0.25g) or [M] or {V-VI}	0.94
(0.26g - 0.35g) or [H] or {VII-VIII}	0.90
(0.36g - 0.50g) or [VH] or {IX-X}	0.85
(> 0.50g) or [EH] or {XI-XII}	0.75

DF_i Dynamic Forces Impact
ERZ Earthquake Risk Zone classifies seismicity to 7 grades as EH (Extremely High), VH (Very High), H (High), M (Moderate), L (Low), VL (Very Low), and EL (Extremely Low); it is listed in the table inside brackets – [ ]
g g-force or peak ground acceleration due to earth's gravity (m/sec^2); 1g = 9.81 m/sec^2
MSK Medvedev-Sponheuer-Karnik Scale (Medvedev and Sponheuer, 1969) classifies seismicity to 12 grades as I to XII; it is listed in the table inside braces – { }
PGA_{SD} Scaled Design Peak Ground Acceleration; it is listed in the table inside parentheses – ( )

PGA_{SD}, ERZ, or MSK are the choices of designer, engineer, or geologist; their values are listed in Table 7 inside parentheses, brackets, and braces respectively.

3.7. Excavation Technique Impact

ET_i is the Excavation Technique Impact (Eq 1 and 8) on the ground-structure behaviour representing vibration impacts on structure during the excavation, which is designed to be a function of Excavation Technique (ET) or Peak Particle Velocity (PPV). ET_i ranges between 1.00 to 0.50 (Figure 9). Table 8 defines values of ET_i.

Table 8. Excavation Technique Impact (ET_i)

(ET) or **[PPV** mm/sec**]**	**ETi**
(ManDigg)	1.00
(ME/NonExBreak) or [< 2]	0.99
(ResiBlast) or [2 - 9]	0.98
(CommBlast) or [10 - 24]	0.97
(IndBlast) or [25 - 59]	0.96
(InfraBlast) or [60 - 119]	0.95
(CtldBlast) or [120 - 449]	0.90
(MineBlast) or [450 - 499]	0.80
(ProdBlast) or [500 - 599]	0.65
(UnCtldBlast) or [≥ 600]	0.50

CommBlast	Commercial Blasting (Engineered blasting near commercial area)
CtldBlast	Controlled Blasting (An ordinary engineered blasting for civil works)
ET	Excavation Technique; it is listed in the table inside parentheses – ( )
ET_i	Excavation Technique Impact
IndBlast	Industrial Blasting (Engineered blasting near industrial area)
InfraBlast	Infrastructures Blasting (Engineered blasting for demolishing the infrastructures)
ManDigg	Manual Digging (Small scale excavation without use of explosives or NonExBreak)
ME	Mechanised Excavation (Medium-large scale excavation without use of explosives or NonExBreak)
MineBlast	Mining Blasting (Controlled blasting with underground/surface mining standards)
NonExBreak	Non-Explosive Breaking (Ground fragmentation using expansive materials)
PPV	Peak Particle Velocity (mm/sec) at the distance of 20 m from blast; listed in the table inside brackets – [ ]
ProdBlast	Production Blasting (Controlled blasting for rock production in large scale)
ResiBlast	Residential Blasting (Engineered blasting near residential area)
UnCtldBlast	Un-Controlled Blasting (Non-engineered blasting)

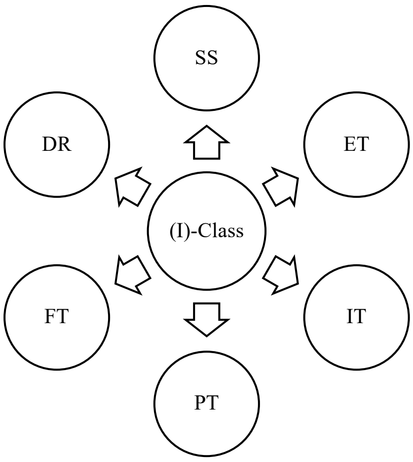
Categorization provided in Table 8 for ET and PPV is based on the research and experience of author (Bineshian, 2019a, 2019b) in design and application of engineered blasting and fragmentation techniques in various strata and several projects; however, AS 2187.2 – 1993 is taken into consideration for PPV limits for engineered blasting near important structure/s. This is for the first time that impact of excavation technique is comprehensively considered in a classification and characterisation system. I-System considers it as an impact factor influencing the total value of (I). If PPV is used as criterion for scoring the ET_i; therefore, it is recommended to measure it using seismographs; however, it can be estimated using empirical relation proposed by United States Bureau of Mines (Duvall and Fogelson, 1962), which is known as USBM PPV Predictor for estimation of blast-induced ground vibration (Appendix 2); otherwise, type of ET is the criterion to pick the proper score for ET_i from Table 8. Vibration-induced Damage (Bineshian, 2021a, 2021b; Appendix 2) assessment is necessary when blasting is used for excavation. Use of ET or PPV in Table 8 for picking the right value for ET_i, is the choice of the designer, engineer, or geologist; it can be used as per availability and condition.

In Table 8, for better differentiation, ET values are listed inside parentheses while PPV values are listed inside brackets.

4. (I)-Class

I-System's Classification entitled "(I)-Class" includes a hexad output, which is illustrated in Figure 11a. Figure 11b is a representation of table of recommendations of (I)-Class that listed below in full form:

- Support System/s (SS)
- Excavation Technique/s (ET)
- Instrumentation/monitoring Technique/s (IT)
- Prevention Technique/s (PT)
- Forecast Technique/s (FT)
- Design Remark/s (DR) to help in structural dimensioning and verification (SD and SV in Figure 2a).



a. (I)-Class output

(I)		Recommended Measure/s					
Range	(I)-Class	SS	ET	IT	PT	FT	DR
100-91	(I)-01						
90-81	(I)-02						
80-71	(I)-03						
70-61	(I)-04						
60-51	(I)-05						
50-41	(I)-06						
40-31	(I)-07						
30-21	(I)-08						
20-11	(I)-09						
10-0	(I)-10						

b. (I)-Class's table of recommendations

Figure 11. I-System's Classification output; (I)-Class

(I) ranges from 100 to 0 (Figures 9 and 11b). (I)-Class classifies the ground into 10 classes as per the value of (I) from (I)-01 as the best to (I)-10 as the worst ground (Figure 11b). Each class has 10 percent share out of 100. Recommendations for SS, ET, IT, PT, FT, and DR are provided for each class in Tables 9 and 10 for underground, semi-surface, and surface structures. Additionally, (I)-Class provides recommendations for special classes (Special (I)-Class) for particular types of ground behaviour/hazards (GB and GH in Figures 2a, 3, and 4) as (I)-BP, (I)-TD, and (I)-VP in Tables 11 and 12. Definition for BP, TD, and VP is recalled here;

BP Burst Prone - ground condition with rock burst or coal burst behaviour
TD Time Dependent - ground condition with time dependent shearing behaviour such as squeezing, swelling, and heaving condition, or even creep
VP Visco-elasto-Plastic - ground condition as visco-elasto-plastic to fully plastic behaviour that contains elastic component/s together with viscous component/s, which gives the ground strain rate dependence on time; however, due to losing energy during static or dynamic loading cycle, its behaviour converts to fully plastic and may flow like a viscous substance.

Furthermore, ET column in Tables 9 and 11 for underground structures provides advice on pull length (PL), which can be estimated using the proposed method in Appendix 3. Nomenclature for all abbreviations used in this section is provided in Section 10.

Table 9. (I)-Class for Underground Structures: SS, ET, IT

(I)	**(I)-Class**	**Recommended Measure/s**		
		SS	**ET**	**IT**
100-91	**(I)-01**	Scaling	FF, ME/DnB, PL	Nil
90 - 81	**(I)-02**	Scaling, IndiB25	FF, ME/DnB, PL	Nil
80 - 71	**(I)-03**	Scaling, SpotB25	FF, ME/DnB, PL	Nil
70 - 61	**(I)-04**	Scaling, SpotB25, PatchPS50	FF, ME/DnB, PL	3DMS@400m
60 - 51	**(I)-05**	Scaling, SpotB32/SysHB25.LS, PS50, PSFS50, RDH54.L	FF, ME/DnB, PL	3DMS@200m
50 - 41	**(I)-06**	Scaling, SysB32.L.S/SysHB32.L.S, FRS100, FRFS50, RDH54.L	HnB/(FF if ≤ 45 m^2), ME/DnB, PL	3DMS@100m, StrainM@300m
40 - 31	**(I)-07**	Scaling, CPS32.L.S/FP32.250.L.X1, SysB32.L.S/SysHB32.L.S, LG25.20.150.1000-, FRS200, FRFS150, RDH54.L	HnB/(FF if ≤ 35 m^2), ME/NonExBreak/ DnB, PL	3DMS@75m, StrainM@250m, PressC/LoadC@300m
30 - 21	**(I)-08**	FP32.200.L.X1/FP76.250.L.X1/ PR100.300.L.X1, SysLB32.L.S, LG32.25.180.1000/ RigidR150UC23.1000-, FRS225/FRC225, FaceButt.L, FRFS200, RDH54.L+CF	PSE, ME/NonExBreak, PL	3DMS@50m, StrainM@200m, PressC/LoadC@250m, SingleRodE@400m
20 - 11	**(I)-09**	PR100.250.L.X1/FP76.200.L.X1/ FP32.200.L.X2, FaceB25.L.S/ FaceP300-, FaceButt.L, PreG/I, RigidR150UC23.750-+RingC, SysN32.L.S, FRS225/FRC225, FRFS200, RDH54.L+CF	PSD, ME, PL	3DMS@25m, StrainM@150m, PressC/LoadC@200m, MultiRodE@400m, StrainG@500m
10 - 0	**(I)-10**	PR100.200.L.X1/FP76.200.L.X2, PreG/I, PostG/I, FaceB32.L.S/ FaceP300-, FaceButt.L, RigidR200UC46.500-+RingC, SysN32.L.S, FRS250/FRC250, FRFS225, (RDH54.L, WDH54.L)+CF	PSD, ME, PL	3DMS@15m, StrainM@100m, PressC/LoadC@150m, MultiRodE@300m, StrainG@400m, DIC@25m

Table 9. Continued; (I)-Class for Underground Structures: PT, FT, DR

(I)	(I)-Class	Recommended Measure/s		
		PT	**FT**	**DR**
100-91	**(I)-01**	Avoid: ‘UnCtldBlast’	TSP/PH100.BH.L	Active load configuration, SPL and/or SFL not required
90 - 81	**(I)-02**	Avoid: ‘UnCtldBlast’	TSP/PH100.BH.L	Active load configuration, SPL and/or SFL not required
80 - 71	**(I)-03**	Avoid: ‘UnCtldBlast’	TSP/PH100.BH.L	Active load configuration, SPL and/or SFL not required
70 - 61	**(I)-04**	Avoid: ‘ProdBlast/UnCtldBlast’	TSP/PH100.BH.L	Active load configuration, SFL not required
60 - 51	**(I)-05**	Avoid: ‘ProdBlast/UnCtldBlast’	TSP/PH100.BH.L/ PH54.EC.L	Load configuration to be maintained as active, SFL not required
50 - 41	**(I)-06**	Avoid: ‘ProdBlast/UnCtldBlast’	TSP/PH100.BH.L/ PH54.EC.L	Load configuration to be maintained as active
40 - 31	**(I)-07**	Apply: ‘CPS’, Avoid: ‘MineBlast/ProdBlast/ UnCtldBlast’	TSP/PH100.BH.L/ PH54.EC.L	Critical load bearing capacity
30 - 21	**(I)-08**	Apply: ‘FP/PR, maintain buttress’, Avoid: ‘FF & DnB’	TSP/PH54.EC.L	Passive load configuration, Sensitive to: ‘scale, unsupported span, & stand-up time’
20 - 11	**(I)-09**	Apply: ‘PreG/I & PR/FP, maintain buttress’, Avoid: ‘FF, NonExBreak/DnB, & ductile SS’	TSP/PH54.EC.L	Passive load configuration, Sensitive to: ‘scale, unsupported span, & stand-up time’
10 - 0	**(I)-10**	Apply: ‘PreG/I & PR, maintain buttress’, Avoid: ‘FF, NonExBreak/DnB, & ductile SS’	TSP/PH54.EC.L	Passive load configuration, Sensitive to: ‘scale, unsupported span, & stand-up time’

Table 10. (I)-Class for Semi-surface and Surface Structures: SS, ET, IT

(I)	(I)-Class	Recommended Measure/s		
		SS	**ET**	**IT**
100-91	**(I)-01**	Scaling	(PreS, DD12000^{-}), (ProdBlast, PD6000^{-})	Nil
90 - 81	**(I)-02**	Scaling, IndiB25	(PreS, DD12000^{-}), (ProdBlast, PD4000^{-})	Nil
80 - 71	**(I)-03**	Scaling, SpotB25	(PreS, DD9000^{-}), (ProdBlast, PD4000^{-})	Nil
70 - 61	**(I)-04**	Scaling, SpotB25/SpotA25, PatchHEAM/PatchWeldM, DH54.L	(PreS, DD9000^{-}), (ProdBlast, PD3000^{-})	3DMS@200m
60 - 51	**(I)-05**	Scaling, SpotB32/SpotA32, HEAM/WeldM, DH54.L	(PreS, DD6000^{-}), (ProdBlast, PD3000^{-})	3DMS@150m
50 - 41	**(I)-06**	Scaling, SysA25.L.S, FRS150, DH54.L	(PreS, DD6000^{-}), (ProdBlast, PD2000^{-})	3DMS@75m, IncM@500m
40 - 31	**(I)-07**	Scaling, SysA32.L.S, FRS250, PostG/I, DH54.L	ME/NonExBreak	3DMS@25m, IncM@400m
30 - 21	**(I)-08**	RWall-SolP/FRS300/FRC300, SysN32.L.S, WH54.L+CF	PSE, ME	3DMS@10m, IncM@300m
20 - 11	**(I)-09**	DWall-TanP/FRS350/FRC350, SysN32.L.S, WH54.L+CF	PSE/OC, ME	3DMS@10m, IncM@200m, DIC
10 - 0	**(I)-10**	DWall-SecP/FRS400/FRC400, SysN32.L.S, WH54.L+CF	PSE/OC, ME	3DMS@10m, IncM@150m, DIC

Table 10. Continued; (I)-Class for Semi-surface and Surface Structures: PT, FT, DR

(I)	**(I)-Class**	**Recommended Measure/s**		
		PT	**FT**	**DR**
100-91	**(I)-01**	Avoid: 'UnCtldBlast'	VPH54.L	Permanent stable condition, SPL and/or SFL not required
90 - 81	**(I)-02**	Avoid: 'UnCtldBlast'	VPH54.L	Check against 'plain failure criteria', SPL and/or SFL not required
80 - 71	**(I)-03**	Avoid: 'UnCtldBlast'	VPH54.L	Check against 'plain/wedge failure criteria', SPL and/or SFL not required
70 - 61	**(I)-04**	Avoid: 'ProdBlast/UnCtldBlast'	VPH54.L	Check against 'plain/wedge failure & rock fall criteria', SPL and/or SFL not required
60 - 51	**(I)-05**	Protect crest with FRS to prevent increment in pore water pressure, Avoid: 'ProdBlast/UnCtldBlast, & bulk removal of toe'	ERT/VPH54.L	Check against 'plain/wedge/toppling failure & rock fall criteria', SFL not required
50 - 41	**(I)-06**	Cover slope crest with WPM & FRS at a width equal to height to help prevention of tension crack generation, Avoid: 'ProdBlast/UnCtldBlast, surcharge at crest, & toe lightening'	ERT/VPH54.L	Check against 'plain/wedge/toppling failure & rock fall criteria'
40 - 31	**(I)-07**	Cover slope crest with WPM & FRS at a width equal to height to help prevention of tension crack generation, Avoid: 'ProdBlast/UnCtldBlast, sharp/tall slope, short berm, surcharge at crest, & toe lightening'	ERT/SRT/VPH54.L	Check against 'plain/wedge/toppling failure & rock fall criteria'
30 - 21	**(I)-08**	Cover slope crest with WPM & FRS at a width equal to height to help prevention of tension crack generation, Avoid: 'NonExBreak/DnB, sharp/tall slope, short berm, & surcharge at crest'	MASW/SRT/ERT/ VPH54.L	Check against 'circular failure criteria'
20 - 11	**(I)-09**	Avoid: 'NonExBreak/DnB, unretained wall/s, & surcharge at crest'	MASW/SRT/ VPH54.L	Check against 'circular failure criteria'
10 - 0	**(I)-10**	Avoid: 'NonExBreak/DnB, unretained wall/s, & surcharge at crest'	MASW/SRT/ VPH54.L	Check against 'circular failure criteria'

Table 11. Special (I)-Class for Underground Structures

(I)-Class	Recommended Measure/s					
	SS	ET	IT	PT	FT	DR
(I)-BP*	Scaling, SysDB25.L.S/ ConeB25.L.S/ YieldB25.L.S, FRS150, SRH100.L.S.X1, HEAM/CableL+ WeldM, FRFS50	HnB, ME/ DnB, PL	3DMS@25m, StrainM@100m, PressC/LoadC@ 300m, MultiRodE@ 600m	Avoid: 'ProdBlast/ UnCtldBlast, rigid SS, & naked faces'	TSP/ PH100. BH.L	Bursting initiation time and depth of plastic zone around periphery to be measured
(I)-TD*	Mild-Severe SSH: YieldR1000+RingC, SRH100+.L.S.X2, YieldFRS200/ YieldFRC200, LSC, SysDB25.L.S Minor SSH: RigidR200UC46.1000 -+RingC, FRS200/FRC200+ SRH100.L.S.X1+ SysLB32.L.S	HnB, ME, PL	3DMS@10m, StrainM@100m, PressC/LoadC@ 150m, MultiRodE@ 300m, StrainG@400m, DIC@25m	Apply: 'SRH, SysLB for Minor SSH' Avoid: 'FF, DnB, rigid SS, & SysLB for Mild-Severe SSH'	TSP/ PH100. BH.L	Nonuniform deformation, load relaxation, scale sensitive
(I)-VP	BulkH300+, FaceP300-, PR100.150.L.X1, PreI/JetG/PreF, PostG/I, RigidR200UC46.500- +RingC, FRS300/FRC300, FRFS275, (RDH54.L, WDH54.L, ADH54.L)+CF	PSD, ME, PL	3DMS@10m, StrainM@100m, PressC/LoadC@ 150m, MultiRodE@ 400m, StrainG@400m, DIC@25m	Apply: 'PreG/I & PR, maintain buttress' Strictly Avoid: 'FF, NonExBreak/ DnB, ductile SS, & build-up of hydrostatic pressure/thrust at face'	TSP/ PH54. EC.L	Passive load configuration, Sensitive to: 'scale, unsupported span, & stand-up time'

* Appendix 4 for further information.

Table 12. Special (I)-Class for Semi-surface and Surface Structures

(I)-Class	Recommended Measure/s					
	SS	ET	IT	PT	FT	DR
(I)-VP	JetG/PreG/I/PreF, DWall-SecP/TanP, WH54.L+CF	PSE/OC, ME	3DMS@10m, DIC	Apply PreG/I/Freezing Strictly Avoid: 'NonExBreak/ DnB, unretained wall/s, & surcharge at crest'	MASW/V PH54.L	Liquefaction prone, vibration sensitive, high passive lateral load configuration in design of retaining structure, long term consideration in time dependent behaviour

Appendix 5 illustrates some of the measures recommended in the SS and FT columns in Tables 9 and 11, including ADH, BH, BulkH, ConeB, CPS, EC, FaceB, FaceButt, FaceP, FibreD, FP, PH, PR, RDH, SysB SysDB, SysHB, SysLB, SysN, WDH, and YieldB. Definition of these measures is presented in Section 10.

Appendix 6 provides systematic bolting calculation method for bolting parameters (length and spacing) for measures proposed in the SS column in Tables 9 and 11, including ConeB, SysB, SysDB, SysLB, SysN, and YieldB. Definition of these measures is presented in Section 10.

5. (I)-GC

I-System's Ground Characterisation entitled "(I)-GC" characterizes the mechanical properties of ground (rock or soil mass) by quantifying most important ground properties including Modulus of Deformation (E_g), Poisson's Ratio (ν_g), Unconfined Compressive Strength (σ_{cg}), Uniaxial Tensile Strength (σ_{tg}), Cohesion (φ_g), and Internal Friction Angle (φ_g). Quantified values provided as output of (I)-GC are estimations based on empirical correlations. Figure 12 is a representation of hexad output for (I)-GC.

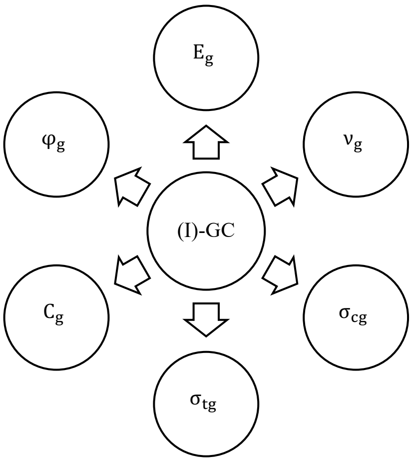


Figure 12. I-System's Ground Characterisation; (I)-GC

(I)-GC's output (Figure 12) provides most important input values required in design approach and procedure (Figures 1 and 2a) for underground, semi-surface, and surface structures. The mathematical form of (I)-GC's hexad output is presented in Eq 11 to 16 (Bineshian, 2019b), whereas the graphical form is presented in Figure 13. These empirical equations are developed and examined by author for several cases (Bineshian, 2019b); however, their accuracy may improve by study on further cases.

$$E_g = e^{0.05\times(I)}-1 \tag{11}$$

$$\nu_g = 0.5 - 0.004 \times (I) \tag{12}$$

$$\sigma_{cg} = 0.007 \times \sigma_c \times e^{0.05\times(I)} \tag{13}$$

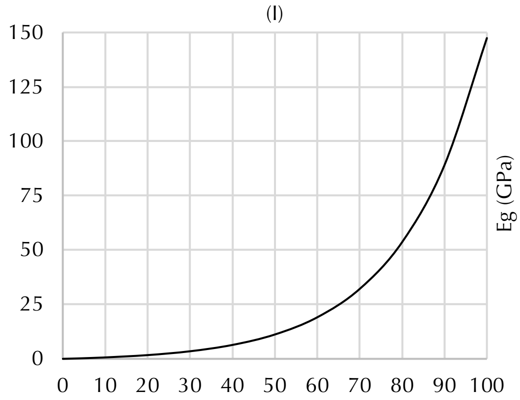
$$\sigma_{tg} = -\sigma_{cg} \times e^{(0.04\times(I)-4)} \tag{14}$$

$$C_g = 0.002 \times \sigma_{cg} \times e^{0.05\times(I)} \tag{15}$$

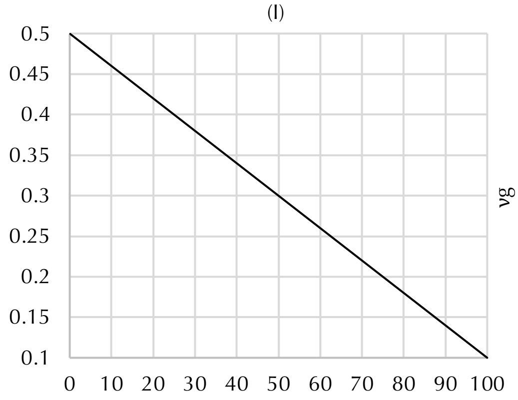
$$\varphi_g = 15 + 0.55 \times (I) \tag{16}$$

where;

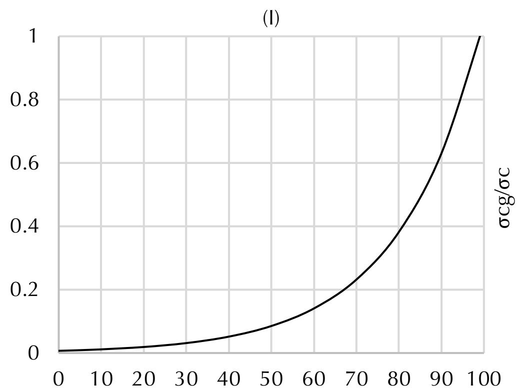
(I)	I-System's Value
E_g	Modulus of Deformation of ground – rock-/soil-mass (GPa)
ν_g	Poisson's Ratio of ground
σ_c	Unconfined Compressive Strength of intact rock or soil (MPa)
σ_{cg}	Unconfined Compressive Strength of ground – rock-/soil-mass (MPa)
σ_{tg}	Uniaxial Tensile Strength of ground – rock-/soil-mass (MPa)
C_g	Cohesion of ground (KPa)
φ_g	Internal Friction Angle of ground (degrees)



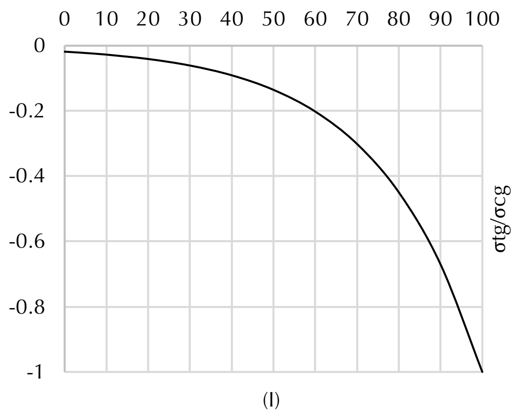
a. (I) vs Modulus of Deformation



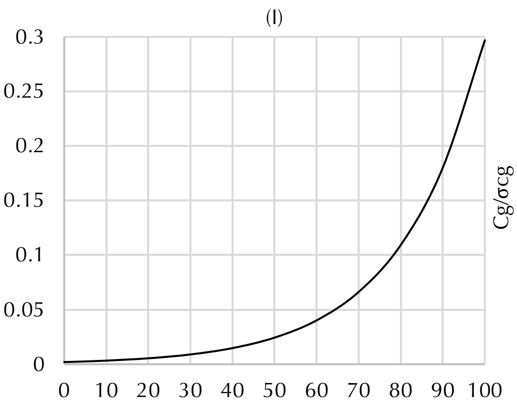
b. (I) vs Poisson's Ratio



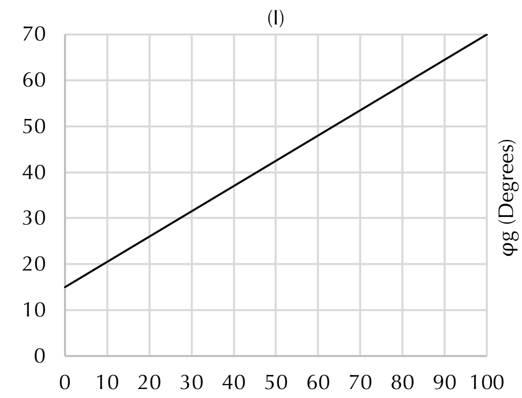
c. (I) vs Unconfined Compressive Strength



d. (I) vs Uniaxial Tensile Strength



e. (I) vs Cohesion



f. (I) vs Internal Friction Angle

Figure 13. (I)-GC Chart

6. Utilisation Guideline

Utilisation approach of I-System is based on the following steps:

- Stage 1. Derivation of input parameters from a site visit or reference data. Figure 2b demonstrates the data group, which is used in I-System as input.
- Stage 2. Calculation of indices; A_i, C_i, H_i, P_i, S_i, DF_i, and ET_i using the derived data in Stage 1, Eq 2 – 8, and Tables 2 – 8.
- Stage 3. Calculation of (I) using Eq 1 and calculated indices in Stage 2.
- Stage 4. Determination of (I)-Class using the calculated (I) value in Stage 3 and Tables 9 – 12. Recommendations for SS, ET, IT, PT, FT, and DR provided in Tables 9 – 12 are applicable in practice.
- Stage 5. Calculation of (I)-GC; E_g, ν_g, σ_{cg}, σ_{tg}, C_g, and φ_g using Eq 11 to 16 or Figure 13, which is applicable for design.

Figure 14 summarises the utilisation approach explained above in a simple diagram.

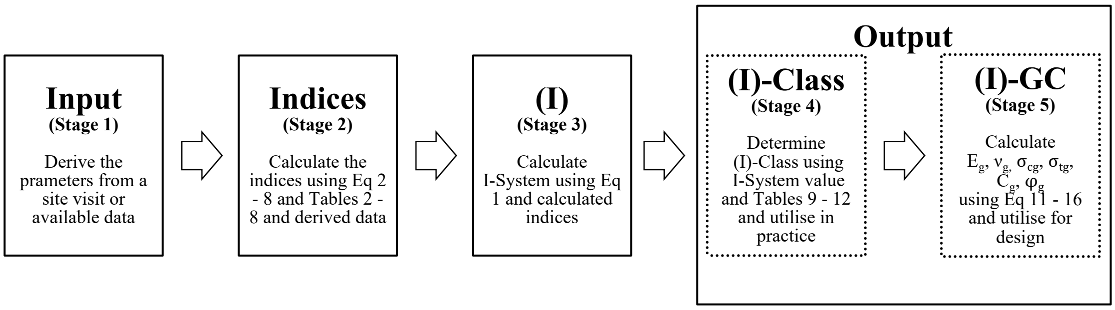


Figure 14. Utilisation diagram of I-System

An example of I-System calculation for a tunnel is provided in Appendix 7; input parameters for calculation of (I) in Figure 24a, (I)-Class as output of classification in Figure 24b, (I)-GC as characterisation output in Figure 24c, and (I)-GC Charts in Figure 24d. Below, a summary of (I)-Class for the same example is provided as a guide for decoding the recommendations' script:

(I) = 25 ⟹ (I)-08 ⟹ Table 9 ⟹ Derive recommendations for (I)-08 as follows:

SS - Support System FP32.200.L.X1/FP76.250.L.X1/PR100.300.L.X1, SysLB32.L.S, LG32.25.180.1000/RigidR150UC23.1000-, FRS225/FRC225, FaceButt.L, FRFS200, RDH54.L+CF
ET - Excavation Technique/s PSE-ME/NonExBreak, PL
IT - Instrumentation Technique/s 3DMS@50m, StrainM@200m, PressC/LoadC@250m, SingleRodE@400m
PT - Prevention Technique/s Apply FP/PR, Maintain Buttress, Avoid: 'FF & DnB'
FT - Forecast Technique/s TSP/PH54.EC.L
Design Remark/s Passive load configuration, sensitive to 'scale, unsupported span, & stand-up time'

Tunnel's largest dimension in a cross section (diameter, width, or height) for above example is 8000 mm; therefore, D = 8000 mm.

Section 10 provides a comprehensive nomenclature that is necessary to be used for decoding of the output of I-System's classification. I-System's classification output, namely, (I)-Class that is provided in Tables 9 – 12, should be decoded using Section 10. Accordingly, above output-example is decoded (using Section 10) and interpreted in details as follows:

- **SS** – Support System to be applied:

  - **PR100.300.L.X1** or **FP76.250.L.X1** or **FP32.200.L.X1**
    (Piperoofing 100 mm dia, 300 mm spacing, L = 0.7D to L = D then L = 5600 to 8000 mm in one row) or
    (Forepoling 76 mm dia, 250 mm spacing, L = 0.7D to L = D then L = 5600 to 8000 mm in one row) or
    (Forepoling 32 mm dia, 200 mm spacing, L = 0.7D to L = D then L = 5600 to 8000 mm in one row).
    Specified length (L) for the piperoofing or forepoling is a function of D (Diameter, width, or height (mm) of underground opening, the greater value), which can be derived from the empirical equation (L = 0.7D to L = D) proposed in Section 10 and Appendix 5. The length between 5600 to 8000 mm to be selected as per condition.

  - **SysLB32.L.S**
    (Systematic Long Bolting 32 mm dia, L = 0.7D or $L = D \times \frac{100-(I)}{100}$ then L = 5600 to 6000 mm, S = 0.3L = 1700 to 1800 mm).
    Specified length (L) and spacing (S) for systematic long bolting as a function of D (Diameter, width, or height (mm) of underground opening, the greater value) can be derived from empirical equations (L = 0.7D and S = 0.3L) proposed in Section 10 and Appendix 5 or using proposed equations in Appendix 6 ($L = D \times \frac{100-(I)}{100}$ and S = 0.3L) as a function of D and (I). The length between 5600 to 6000 mm to be selected as per condition.

  - **LG32.25.180.1000$^-$** or **RigidR150UC23.1000$^-$**
    (Lattice Girder with 32 mm dia rebar at intrados and two 25 mm dia rebars at extrados with 180 mm spacing between the intrados and extrados and spacing between the LGs below 1000 mm) or
    (Rigid Rib made with Universal Column as per Australian Standard of 150UC23 and spacing of below 1000 mm).

  - **FRS225** or **FRC225**
    (Fibre Reinforced Shotcrete with 225 mm thickness) or
    (Fibre Reinforced Concrete with 225 mm thickness).

  - **FaceButt.L**
    (Face Buttress with L = 0.25D = 2000 mm if D ≥ 6000 mm).
    Specified length (L) for buttress as a function of D (Diameter, width, or height (mm) of underground opening, the greater value) can be derived from empirical equation (L ≥ 0.25D) proposed in Section 10 and Appendix 5.

  - **FRFS200**
    (Fibre Reinforced Face Sealing with 200 mm thickness).

  - **RDH54.L+CF**
    (Radial Drainage Holes 54 mm dia, L ≤ D ≅ 8000 mm + Collar Filtration).
    Specified length (L) for radial drainage holes as a function of D (Diameter, width, or height (mm) of underground opening, the greater value) can be derived from empirical equation proposed in Section 10 and Appendix 5 (L ≤ D).

- **ET** – Excavation Technique/s to be implemented:

    - **PSE-ME/NonExBreak, PL**
      (Partial Sequential Excavation using Mechanised Excavation or Non-Explosives Breaking with Pull Length as PL $= 0.5\mathrm{D}\frac{(\mathrm{I})}{100}$ then PL = 1000 mm).
      Specified pull length (PL) for advance at face as a function of D (Diameter, width, or height (mm) of underground opening, the greater value) and (I) can be derived from empirical equation (PL $= 0.5\mathrm{D}\frac{(\mathrm{I})}{100}$) proposed in Appendix 3.

- **IT** – Instrumentation Technique/s to be used:

    - **3DMS@50m**
      (3D Monitoring Station at every 50 m).

    - **StrainM@200m**
      (Strain Meter at every 200 m).

    - **PressC/LoadC@250m**
      (Pressure Cell or Load Cell at every 250 m).

    - **SingleRodE@400m**
      (Single-Rod Extensometer at every 400 m).

- **PT** – Prevention Technique/s to be considered:

    - Apply **PR/FP** (Piperoofing or Forepoling).

    - Maintain **Buttress**.

    - Avoid '**FF** (Full Face Excavation) and **DnB** (Drill and Blast)'.

- **FT** – Forecast Technique/s to be utilised:

    - **TSP/PH54.EC.L**
      (Tunnel Seismic Prediction) or
      (Probe Hole 54 mm dia using Exploratory Coring with L = 3D = 24000 mm).
      Specified length (L) for probe hole using exploratory coring as a function of D (Diameter, width, or height (mm) of underground opening, the greater value) can be derived from empirical equation (L = 3D) proposed in Section 10.

- **DR –** Design Remark/s to be taken into consideration:

    - Passive load configuration, and

    - Sensitive to 'scale, unsupported span, and stand-up time'.

Example provided here in this section is analysed with I-System Software (it is introduced in Section 7 and output of the same is presented in Appendix 7). Further case studies are available in reference number 8 for application of I-System for underground, semi-surface, and surface structures.

Notably, two methods are used for calculation of the length of the Systematic Long Bolting (SysLB), which illustrated in Appendix 5 and fully explained in Appendix 6. It is the choice of designer, engineer, or geologist to select the method that is more compatible with site condition; however, it is recommended to take the greater value between the calculated ones (using illustrations in Appendix 5 or proposed equation in Appendix 6). Same logic is applicable for other calculations for the proposed measures.

7. I-System Software

A software for I-System is developed in 2020 named "I-System Software" aimed to ease the use of I-System and to ensure high accuracy and precision in calculation procedure for classification as well as characterisation is obtained.

I-System Software uses the same algorithm of I-System (Bineshian, 2019a, 2019b, 2020c) originally published and it works exactly as per the I-System principle using the same formulations, tables, and approaches for classification as well as characterisation of ground in relation to underground, semi-surface, and surface structures.

I-System Software works as per following flowchart (Figure 15):

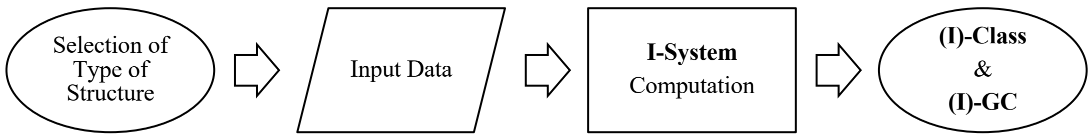


Figure 15. Computation flowchart of I-System Software

- Type of structure includes underground, semi-surface, or surface in which the classification and characterisation is going to be conducted for.
- Input data includes the same input data that considered for hand-calculation of I-System (Stage 1 of Section 6). Appendix 7 (Figure 24a) represents a print of the input of the software.
- Computation includes the Stages 2 and 3 of Section 6, which is the calculation of indices and consequently (I).
- The output includes (I)-Class and (I)-GC, which is the same as Stages 4 and 5 of Section 6. Appendix 7 (Figures 24b and 24c) represent print of the output of the I-System Software.

(I)-Class's output (Section 4) includes hexad of SS, ET, IT, PT, FT, and DR that are applicable in practice. The same is the output of the software for (I)-Class shown in Appendix 7 (Figure 24b). (I)-GC's output (Section 5) includes hexad of $\mathrm{E_g}$, ν_g, σ_{cg}, σ_{tg}, $\mathrm{C_g}$, and φ_{g} in form of values and chart that are applicable in design as input. Appendix 7 (Figure 24c) shows the same as output of the software for (I)-GC. Besides, I-System Software provides additional output, namely, (I)-GC Chart that is the graphical representation of (I)-GC (Appendix 7; Figure 24d).

Additionally, the software provides GCD calculator that can be used for measurements of ground hydraulic conductivity that is used as input in the software for $\mathrm{H_i}$ (Hydro Index) or it may be used individually in practice and/or in design for grouting/injection assessment (Bineshian, 2020a). Also, other utilities included in the software that are helpful for a complete classification and characterisation of ground. Summarily, output report or print of the software (Appendix 7) contains full details of input data (Figure 24a); entered by user and processed by the software), (I)-Class details (Figure 24b); computed by the software), and (I)-GC details and charts (Figures 24c and 24d); calculated and plotted by the software). Appendix 7 at the end of this paper provides output of the software for the case, which is solved and decoded in Section 6. I-System Software is an engineering utility for classification and characterisation of ground; however, it is under further development for more applications in design and practice.

8. Conclusions

I-System is developed to compensate demerits of existing engineering classifications including their limitations, drawbacks, impreciseness, and inaccuracy. It is applicable for rock and soil with acceptable precision and accuracy with simplicity in use and certainty in its approach for derivation of input parameters besides clarity and trust in the output data.

It is developed in challenging projects in varieties of ground and verified for perfectness. There is no limitation/s in its application for any type of underground, semi-surface, and surface structures in rock and soil. It comes with a simple equation containing essential parameters, which can be derived from doubtless input tables, reliable references, or test results. It is based on certain key indices, which defines mechanical behaviour of surrounding ground of structure considering impact of dynamic forces as well as excavation technique impact.

I-System contains two main parts; (I)-Class as classification system and (I)-GC as characterization system.

(I)-Class classifies the ground to 10 classes from the best to the worst, which contains a hexad output as recommendations that is required in practice for execution including; Support System, Excavation Technique/s, Instrumentation Technique/s, Prevention Technique/s, Forecast Technique/s, and Design Remark/s.

(I)-GC provides a hexad output, which is required for design as input including; E_g, ν_g, σ_{cg}, σ_{tg}, C_g, and φ_g.

I-System practically takes into consideration the most important mechanical aspects of ground for an appropriate optimised design. It has the capacity to be a reliable comprehensive classification as well as characterisation system to be utilised in both practice and design for all ground related structures.

9. Future Research Recommendations

Author recommends following additional researches for further development and improvement of I-System:

- Scrutinization of indices including their parameters as well as their scorings for a better modelling of each index of ground.
- Investigation on impact factors and their influence on total value of I-System to obtain better accuracy – if possible – in effect of influencing parameters on (I).
- Study on impact of important factors of shape, scale, in-situ stresses, and/or overburden, which are already considered in Strength Index to develop – if possible – a more effective method in consideration of these factors.
- Adding more recommendation/s on required SS, ET, IT, PT, FT, and DR as output of (I)-Class.
- Work on (I)-GC and their output in characterisation of mechanical aspects of ground for already proposed properties and more parameters by collecting further data in a comprehensive range and varieties of ground for obtaining best fit to derive more accurate correlations.

10. Nomenclature

(I)	I-System's value
(I)-Class	I-System's Ground Classification, which provides recommendations on SS, ET, IT, PT, FT, and DR that are applicable in practice as well as design for structures in ground
(I)-GC	I-System's Ground Characterisation, which provides hexad of E_g, ν_g, σ_{cg}, σ_{tg}, C_g, and φ_g that are applicable in design as input parameters for design of structures in ground
3DM	3D Monitoring - using Bi-Reflex Target
3DMS	3D Monitoring Station
a_{da}	Factor related to "Discontinuity Aperture" that is based on the most unfavourable opening of the discontinuities; a parameter of A_i
a_{dd}	Factor related to "Discontinuity Disintegration" that is based on the worst weathering or alteration of surface of the discontinuity sets; a parameter of A_i
a_{df}	Factor related to "Discontinuity Friction" that is based on the least friction condition of discontinuity sets; a parameter of A_i
ADH	Axial Drainage Hole/s - NX hole/s (w/- or w/o casing), parallel to axis of tunnel, perpendicular to face; $L \leq 1.5D$, S as per site condition
a_{di}	Score related to "Discontinuity Inclination" that is based on the dip angle of the most critical/unfavourable discontinuity set; a parameter of A_i
a_{dn}	Score related to "Discontinuity Number/s" that is based on number of individual discontinuities per meter of a horizontal or vertical scanline or average of number of discontinuities per meter of horizontal and vertical scanline; a parameter of A_i
a_{dp}	Factor related to "Discontinuity Persistency" that is based on the most unfavourable discontinuity set; a parameter of A_i
a_{ds}	Score related to "Discontinuity Set/s" reflecting the number of sets of discontinuities; a parameter of A_i
A_i	Armature Index
B	Width of a berm in a slope or trench or width or horizontal span of an underground space
B/H	Underground, semi-surface, or surface structures' shape or scale factor as ratio of horizontal span to height of underground opening or ratio of width of berm to height of slope or trench
BH	Blind Hole - triangular patterned probing parallel to axis of underground space using blind hole/s; L = 2D and 100+ mm diameter
BP	Burst Prone – highly stressed ground condition with rock burst or coal burst behaviour
BRT	Bi-Reflex Target - 3, 5, or 7 targets installed in a 3DMS based on severity of convergency
BulkH	Bulk Head - shotcrete/concrete plug at whole section of excavation at face to prevent the ground from flowing; $L \leq 0.15D$
C	Convergency (mm)
c/a	Conditionally Applicable
c_1	Site constant in USBM PPV Predictor
c_2	Site constant in USBM PPV Predictor
CableL	Cable Lacing - applicable for controlling rock burst in deep underground spaces
CF	Collar Filtration - filtration of drainage holes' outlet to stop debris/fines discharge
C_g	Cohesion of ground (MPa)
C_i	Configuration Index
CommBlast	Commercial Blasting (engineered blast near commercial area)
ConeB	Cone Bolts - oriented/radial cone bolts; L = 0.5D, S = 0.3L

Continuum Massive Rock	A massive medium rather than layered one; e.g., intact rock or unlayered and structurally interlocked rock mass
c_{pc}	Impacting factor related to "Problematical Configuration" of ground indicating ground's tectonic state; a parameter of C_i
CPD_{max}	Maximum charge per delay (kg)
CPS	Crown Periphery Spiling - SN umbrella at 5-30 deg; L = 0.7D
c_{sc}	Score of "Structural Configuration" of ground (an effect of ground's texture, fabric, and structure); a parameter of C_i
CtldBlast	Controlled Blasting (an ordinary engineered blast for civil works)
CYSS	Conventional Yield Support System - a conventional system of yield measures used in tunnelling under SSH condition; it includes TH or H sliding ribs, LSC, and/or LCN
d	Depth of placement of the structure
D	Diameter, width, or height (mm) of underground opening (the greater value)
DD	Drilling Depth (mm)
DF_i	Dynamic Forces Impact
DH	Drainage Hole/s - upward NX hole/s (w/- or w/o casing); L = 1.5H, S as per site condition
Di	Damage Indicator (%); Bineshian (2021a, 2021b)
Di-Class	Classification of ViD based on Di
DIC	Digital Image Correlation (Bineshian et al, 2021a, 2021b)
Dist	The distance between the blasting location and concerned structure (m)
DL	Drilling Length of blastholes (mm)
DnB	Drill and Blast - controlled blast using a designed drilling pattern
DR	Design Remark/s
DWall	Diaphragm Wall
EC	Exploratory Coring - single NX hole parallel to axis of tunnel; L = 3D
E_g	Deformation Modulus of ground - rock mass or soil mass's deformation modulus (GPa)
ElFootR	Elephant Foot Rib – a stiff/rigid rib applicable when vertical load above crown is high
EN	European Standard – Eurocode of practice
ERT	Electrical Resistivity Tomography - a non-destructive geophysical method for ground characterisation
ERZ	Earthquake Risk Zone classifies seismicity to EH (Extremely High), VH (Very High), H (High), M (Moderate), L (Low), VL (Very Low), EL (Extremely Low)
ET	Excavation Technique/s
ET_i	Excavation Technique Impact
FaceB	Face Bolting – Fibreglass Dowel/s or SDA bolts drilled parallel to axis to support against face pressure/thrust, perpendicular to face; L = D, S ≤ 0.3L
FaceButt	Face Buttress - keeping part of face in place as a buttress to absorb face pressure or thrust as part of face stabilization; L ≥ 0.25D (Only if D ≥ 6 m)
FaceP	Face Plug - shotcrete at face to plug outlet of debris discharge; L ≤ 0.05D
f_b	Frequency of blast-induced vibration (Hz)
FF	Full Face excavation
FibreD	Fibreglass Dowel/s - used as FaceB; L = D, S ≤ 0.3L
FP	Fore Poling - umbrella using perforated/blind SDA; L = D
FRC	Fibre Reinforced Concrete
Freezing	A pre-excavation solidification for underground, semi-surface, and surface openings
FRFS	Fibre Reinforced Face Sealing

FRS	Fibre Reinforced Shotcrete
FT	Forecast Technique/s
g	g-force or peak ground acceleration due to earth's gravity (m/sec^2); 1g = 9.81 m/sec^2
GB	Ground Behaviour based on mechanical response of ground
GCD	Ground Conductivity Designation (Bineshian, 2020a)
GCD_e	Existing GCD; post-blast measured GCD
GCD_p	Pre-existing GCD; pre-blast measured GCD
GC_{ef}	Ground Conductivity Enhanced Factor (Bineshian, 2021a, 2021b)
GD	Gravity Driven - flowing ground with fully plastic behaviour
GH	Ground Hazards based on failure categorisation
Rock	Intact rock or rock mass
Granular	Soil mass (conglomerate and breccia is excluded from this category)
GRC	Ground Reaction Curve
GZ	Ground Zoning based on ground properties
H	Height of a slope, trench, opening, or buttress
HCF	Half Cast Factor (%)
HEAM	High Energy Absorption Mesh - mesh over shotcrete; protective mesh against dynamic or impact loads
h_{gc}	Score assigned to "Ground Conductivity" that is measured using GCD or selected from Wetness diagram as criterion for hydropressure effect on ground; a parameter of H_i
h_{gs}	Impact factor related to "Ground Softness" that is considered as an effect of water on medium/infilling material (Mohs); a parameter of H_i
H_i	Hydro Index
HnB	Heading and Benching - an excavation method to control the scale effect on stability
I-System	Index of Ground-Structure; a comprehensive classification and characterisation system for ground including both rock and soil media (Bineshian, 2019a, 2019b, 2020c)
IncM	Inclinometer/s
IndBlast	Industrial Blasting (engineered blast near industrial area)
IndiB	Individual Bolting - oriented and in very limited number
InfraBlast	Infrastructures Blasting (engineered blast for demolishing infrastructures)
IS	Indian Standard - code of practice
IT	Instrumentation Technique/s
JetG	Jet Grouting - applicable in construction of underground, semi-surface, and surface metro station
Jointless	A definition describing an important feature of intact rock; a medium that does not have any countable joint set
L	Length of ADH, BH, ConeB, CPS, DH, EC, FaceB, FaceButt, FaceP, FP, PH, PR, RDH, SRH, SysA, SysB, SysDB, SysHB, SysLB, SysN, VPH, WDH, WH, and YieldB (mm)
LCN	Longitudinal Compression Niche; applicable in YSS for tunnelling under SSH condition (Bineshian, 2020b)
L_{ch}	Length of contour or periphery blasthole (m)
LG	Lattice Girder
L_{hc}	Length of half-cast or half barrel (m)
L_i	Length of water injected portion (packed length) of drilled hole (m) or length of casing hole (m) or length of installed perforated SDA (m) in the GCD test procedure
LoadC	Load Cell/s
LRFD	Load and Resistance Factor Design method

LSC	Longitudinal or Liner Stress Controller - rubber/spring/soft-timber/rolled-MSP; it is a member of CYSS for tunnelling under SSH condition (Bineshian, 2020b)
LSD	Limit State Design method
M	Earthquake Magnitude
ManDigg	Manual Digging (small scale excavation without use of explosives or NonExBreak)
MASW	Multichannel Analysis of Surface Waves - a non-destructive geophysical method for characterisation of ground
McNally	A system for rock burst treatment in tunnelling using TBM
ME	Mechanised Excavation (Medium- to large-scale excavation using TBM, Roadheader, Excavator, or Hammer without use of explosives or NonExBreak)
MicroP	Micro Piles - distribute concentrated load to a wider footing area under elephant ribs
MineBlast	Mining Blasting (controlled blast as per mining standards)
MSF	Magnitude Scaling Factor
MSK	Medvedev-Sponheuer-Karnik Scale classifies seismicity as I to XII
MSP	Mild Steel Plate
MultiRodE	Multiple Rod Extensometer - measuring points @ 2, 4, and 6 m recommended
n/a	Not Applicable
NATM	New Austrian Tunnelling Method; it minimises SS needs based on utilisation of ground capacity in load bearing and activation of load configuration by application of active SS. NATM is applicable for comprehensive range of ground conditions. SCL and SEM are following the same philosophy of NATM.
NMT	Norwegian Method of Tunnelling; it is a method in tunnelling using Q for ground classification and cross-hole seismic tomography for further characterisation. NMT is not providing a new philosophy in tunnelling.
NonExBreak	Non-Explosive Breaking (ground fragmentation using expansive materials)
NX	Hole with 54.7 mm diameter
OB	Over Break or over cut or over excavation
OC	Open Cut
PatchHEAM	Patch High Energy Absorption Mesh (protection against dynamic/impact loads) – used in slope protections against rock falls and/or tunnelling under burst prone condition
PatchPS	Patch Plain Shotcrete
PatchWeldM	Patch Weld Mesh - applicable as protective mesh in underground, semi-surface, and surface openings to prevent spot rock falls
p_{bw}	Factor related to "Body Wave Velocity" including Vp or Vs as geophysical properties of ground that corrects P_i; Body Wave Velocity is derived either from reliable references (considering the type of materials of ground) or is measured using geophysical methods
p_{cc}	Score related to "Cohesiveness Consistency" that is an important shear property of soil (cohesion); a parameter of P_i
PCC	Plain Cement Concrete
PD	Pull Depth
p_{dc}	Score related to "Denseness Consistency" that is an important shear property of soil (non-cohesiveness; friction); a parameter of P_i
PGA	Peak Ground Acceleration (g)
PGA_{SD}	Scaled Design Peak Ground Acceleration (g); desired scaled PGA
PH	Probe Hole - probing using blind hole drilling with 100+ mm diameter or exploratory coring using NX hole/s; L = 2D for BH and L = 3D for EC
P_i	Properties Index
PL	Pull Length (mm) - advance length

P_m	Peak head (MPa) during injection period of T_i in GCD test procedure; it is the measured water pressure before the first drop in peak is observed.
PostG/I	Post-excavation Grouting/Injection - consolidation/solidification
p_{pm}	Influencing parameter related to "Particles' Morphology" that is a function of shape of soil's grains/granules; a parameter of P_i
p_{ps}	Influencing parameter related to "Particles' Size" that is a function of size of soil's grains/granules"; a parameter of P_i
PPV	Peak Particle Velocity (mm/sec)
PR	Pipe Roofing - perforated/blind pipe (w/- or w/o grouting); L = D
PreF	Pre-Excavation Freezing of face or excavation line/periphery
PreG/I	Pre-excavation Grouting/Injection - cement/mineral/chemical-base
PreS	Pre-excavation Splitting
PressC	Pressure Cell/s
ProdBlast	Production Blasting (controlled blast for rock production in large scale)
PS	Plain Shotcrete
PSD	Partial-Sequential Digging - small scale partial digging in several sequences e.g., small pilots, considering stand-up time and maximum unsupported span
PSE	Partial-Sequential Excavation - small scale partial excavation larger than digging scale in several sequences e.g., pilot and enlargement, considering stand-up time and maximum unsupported span
PSFS	Plane Shotcrete Face Sealing - application of 50 mm plain shotcrete at face to prevent hazards/disintegration
PT	Prevention Technique/s
PU-2C	Polyurethane with two components
Q	Rock mass classification for tunnel supports (Barton et al, 1974)
Q_w	Water intake rate (lit/min) in GCD test procedure
RCC	Reinforced Cement Concrete (Conventional)
RDH	Radial Drainage Hole/s - NX radial holes (w/- or w/o casing); L $\leq$ D, S as per site condition
ResiBlast	Residential Blasting (engineered blast near residential area)
RigidR	Rigid Ribs – steel ribs made from H profile (heavy beam) or any equivalent profile/s used for manufacturing rigid ribs to absorb entire dead/passive load from the ground
RingC	Ring Closure or invert closure
RMR	Rock Mass Rating (Bieniawski, 1973)
RSS	Rigid Support System
RWall	Retaining Wall including cladding wall and any other types
S	Spacing related to ConeB, CPS, FaceB, SRH, SysA, SysB, SysDB, SysHB, SysLB, SysN, or YieldB; S $\leq$ 0.3L
SCL	Sprayed Concrete Lining; it is a tunnelling method using shotcrete (sprayed concrete) as a primary liner as a member of SS for interaction with ground load configuration. SCL is almost the same as NATM with higher emphasis on shotcreting as primary SS.
s_{cs}	Score related to "Compressive Strength" as Uniaxial Compressive Strength of ground; a parameter of S_i
SD	Structural Dimensioning for each SS
SDA	Self-Drilling Anchor
SecP	Secant Piling or equivalent driven or bored piles including friction or end bearing piles
SEM	Sequential Excavation Method; it is not a tunnelling method; instead, it is an excavation method based on NATM principles for optimisation of load configuration.

SF	Scaling Factor
SFL	Structural Final Liner
S_i	Strength Index
SingleRodE	Single Rod Extensometer - measuring point @ 3 m recommended
SLS	Serviceability Limit State design check - a LSD method
SN	Store Norfors - a rigid system of bolts using steel rebars
SolP	Soldier Piling or equivalent driven or bored piles
SPL	Structural Primary Liner
SpotA	Spot Anchoring
SpotB	Spot Bolting - oriented with limited number
SRH	Stress Release Holes - long radial naked holes; 100 - 300 mm diameter; L = D (Bineshian, 2020b)
SRT	Seismic Refraction Tomography - a non-destructive geophysical method
SS	Support System
s_{se}	“Scale Effect” factor; a parameter of S_i
SSH	Squeezing/Swelling/Heaving (Bineshian, 2020b)
SSH-Class	Classification for SSH ground based on severity of convergency (Bineshian, 2020b)
StrainG	Strain Gauge/s
StrainM	Strain Meter
SurS	Surface Structure including surface and semi-surface structure/s and mine/s in general comprising of but not limited to bridge and dam abutments, cut & cover, deep and shallow foundations, embankment and tailing dams, open cuts, open pits, shallow metro stations (cut & cover or open cut), slopes, surface power house openings, trenches
SV	Structural Verification based on the definition of relative safety margin for SD
Swellex	An expandable rock bolting system from Atlas Copco
SysA	Systematic Anchoring - anchors perpendicular to face of slope; L = 0.5H, S = 0.3L
SysB	Systematic Bolting - radial direction; L = 0.5D, S = 0.3L
SysDB	Systematic Dynamic Bolts - oriented/radial dynamic bolts; a system of ductile bolting, which is applicable for tunnelling under stressed condition like burst prone and/or SSH ground (Bineshian, 2020b); L = 0.5D, S = 0.3L
SysHB	Systematic Horn Bolting – a system of bolts oriented towards the direction of advancement at tunnel, which is used to prevent over-breaks and rock falls from crown in tunnelling with unfavourable orientation of discontinuities; to be used only above SPL of tunnel (crown) at 30 - 45 deg; L = 0.7D, $S \leq 0.3L$
SysLB	Systematic Long Bolting - radial long bolts; L = 0.7D, S = 0.3L
SysN	Systematic Nailing - radial bolts/anchors; L = D = H, S = 0.3L
t	Time period or duration of vibration in a blast (sec)
TanP	Tangent Piling or equivalent driven or bored piles including friction or end bearing piles
TBM	Tunnel Boring Machine
TD	Time Dependent - ground condition with time dependent shearing behaviour such as squeezing/swelling/heaving condition, or even creep
TH	Toussaint-Heintzmann - steel profile used in fabrication of yield/sliding ribs
T_i	Injection period (minutes) taken for injection of V_w quantity of water in GCD test procedure; it is the period of time from initial raise in pressure till the first drop in peak.
TSP	Tunnel Seismic Prediction
UB	Under Break or under cut or under excavation
UC	Universal Column as per Australian Standard (i.e., 150UC23 and 200UC46)

UCS	Unconfined Compressive Strength
ULS	Ultimate Limit State design check - a LSD method
UnCtldBlast	Un-Controlled Blasting (Non-engineered blast)
UndS	Underground Structure/s including underground shallow and deep structures, openings, and mines comprising of but not limited to caverns, deep metro stations, galleries, stopes, shafts, tunnels, underground power houses, stations, storages, wells
USBM	The United States Bureau of Mines
ViD	Vibration-induced Damage or blast-induced damage (Bineshian, 2021a, 2021b)
Vp	Primary Wave Velocity (m/sec)
VP	Visco-elasto-Plastic - ground condition as visco-elasto- to fully plastic behaviour; ground contains elastic component/s together with viscous component/s that causes strain rate dependence on time; however, due to losing energy during static or dynamic loading cycle, its behaviour converts to fully plastic and may flow like viscous substance.
VPH	Vertical Probe Hole - vertical NX blind/coring exploration hole/s; L = 0.5H
Vs	Shear/Secondary Wave Velocity (m/sec)
V_w	Injected quantity of water (lit) during injection period of T_i in GCD test procedure; it is measured from the moment that the pressure is started rising till the first drop in peak is observed.
w	Width of crack in concrete (mm); as per IS 456:2000 permissible crack width in the SLS design check for SV must be as: $w < 0.30$ mm
WDH	Wing Drainage Holes - NX wing shape (w/- or w/o casing) at 30 - 45 deg applicable in underground openings to drain water from sides and ahead of face to reduce the pore hydrostatic pressure; $L \leq 2D$, S as per site condition
WeldM	Weld Mesh - conventional weld mesh used over shotcrete in ground burst condition or used as reinforcement for shotcrete
Wetness	A diagram defined here to clarify the ground's water content, which is classifying the ground water condition (observational identification) in 11 ranges
WH	Weep Holes - upward angled NX weeps (w/- or w/o casing); L = H, S as per condition
WPM	Waterproofing Membrane - an elastic/flexible impermeable geotextile or fibre reinforced geomembrane or composite to be used for sealing
X1	One Row
X2	Two Rows
YieldB	Yielding Bolts - oriented/radial ductile bolts for stressed condition including burst prone and/or SSH ground (Bineshian, 2020b); L = 0.5D, S = 0.3L
YieldFRC	Yield Fibre Reinforced Concrete - FRC with embedded LSC and/or LCN, which is applicable for tunnelling under SSH condition (Bineshian, 2020b)
YieldFRS	Yield Fibre Reinforced Shotcrete - FRS with embedded LSC and/or LCN, which is applicable for tunnelling under SSH condition (Bineshian, 2020b)
YieldR	Yield Ribs - sliding ribs using TH or H yield/sliding steel profile; it is an important member of CYSS for tunnelling under SSH condition (Bineshian, 2020b)
YSS	Yield Support System - a system of yield measures, which is used for tunnelling under SSH condition; it includes CYSS and/or SRH System (Bineshian, 2020b)
ν_g	Poisson's Ratio of ground
σ_c	Unconfined Compressive Strength of intact rock or soil (MPa)
σ_{cg}	Unconfined Compressive Strength of ground - rock mass or soil mass (MPa)
σ_h	Horizontal Stresses (MPa) at the location or depth of placement of structure (d)
σ_{tg}	Uniaxial Tensile Strength of ground - rock mass or soil mass (MPa)
σ_v	Vertical Stresses (MPa) at the location or depth of placement of structure (d)
φ_g	Internal Friction Angle of ground (degrees)

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Appendix 1: GCD

Ground Conductivity Designation (GCD) is a test method based on a simple single stage water injection procedure for examination of the ground's hydraulic conductivity (Bineshian, 2020a). The output of GCD guides engineers and/or geologists to have a pre- and/or post-grouting/injection assessment on ground quality in terms of permeability, solidification, consolidation, water ingress reduction, or sealing quality.

Eq 17 represents dimensionless empirical form of GCD. Eq 18 represents water intake rate in lit/min, which is used in Eq 17. Figure 16 demonstrates schematics for the GCD test setup. Table 13 provides classification for ground hydraulic conductivity as well as ground solidification quality.

$$\mathrm{GCD} = \mathrm{Q_w}/(\mathrm{P_m} + \mathrm{L_i}) \tag{17}$$

$$\mathrm{Q_w} = \frac{\mathrm{V_w}}{\mathrm{T_i}} \tag{18}$$

where;
GCD Ground Conductivity Designation (dimensionless)
L_i Length of water injected portion (packed length) of hole or perforated SDA (m)
P_m Peak head (MPa) during injection period of T_i; measured pressure before the first drop in peak
Q_w Water intake rate (lit/min); to be calculated using Eq 18
T_i Injection period of time (min) taken to inject V_w quantity of water; it is the period of time from initial raise in pressure till the first drop in peak is observed.
V_w Injected quantity of water (lit) during injection period of T_i; it is measured from the time that pressure is started to raise till the first drop in peak pressure is observed.

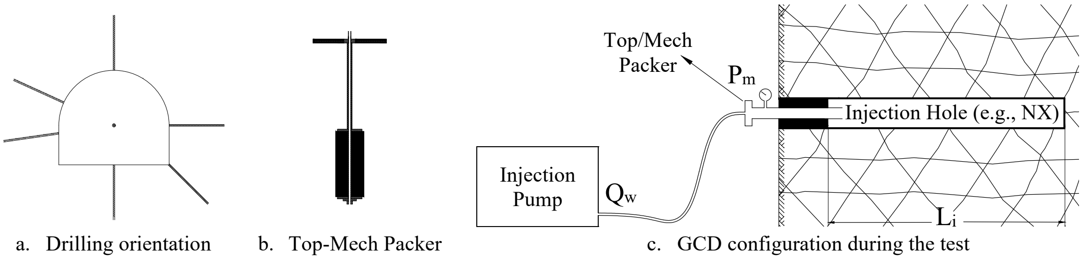


Figure 16. GCD setup (Bineshian, 2020a)

Table 13. Ground Conductivity Designation (Bineshian, 2020a)

Ground Hydraulic Conductivity	GCD	Ground Solidification Quality
Very High - VH	> 100	VP - Very Poor
High - H	100 - 51	P - Poor
Medium - M^+	50 - 16	F - Fair
Moderate - M^-	15 - 6	G - Good
Low - L	5 - 1	VG - Very Good
Very Low - VL	< 1	E - Excellent

A proper conduction of GCD test method includes procedures as follows:

1. Select the location in which ground conductivity to be measured.

2. Drill a single naked hole in any direction or orientation in the chosen location including horizontal, vertical, or inclined at face, wall/s, crown, or invert (Figure 16a). Drilling can be conducted using a rotary-percussion or rotary drilling system; however, rotary drilling system is preferred.

3. Stabilise the drilled hole using casing; however, naked hole for GCD test is preferred. If hole is not sustained, casing can be applied or SDA can be used.

4. Flush the naked hole using clear water to remove fine debris and cuttings. If casing hole or SDA is used, the same flushing procedure to be applied.

5. Pack the collar of the naked drilled hole using a top/mechanical packer (Figure 16b). Packing must be conducted in a proper way that the collar is completely sealed and no leakage of water is observed. If casing or SDA is used, a proper packing inside casing or on the outlet of SDA is necessary to be conducted. The space between casing and ground and/or between the SDA and ground should be completely sealed only at collar using cement mortar or PU-2C or any method/material that may be applicable. It is important to note that only a short portion (maximum 300 mm length) at collar of the hole to be sealed.

6. Set up a suitable water pump and connections for injection of water to the hole (Figure 16c). The water pump should have the capacity in providing enough pressure and be equipped with pressure gauge. Use of grouting pump unit in GCD test procedure is highly recommended.

7. Inject water to the hole and measure the V_w in T_i period of time.

8. Calculate Q_w and consequently GCD.

9. Find the proper range of GCD in Table 13 based on the calculated value.

10. Classify the Ground Hydraulic Conductivity and Ground Solidification Quality using Table 13 for further judgment and use in design and/or practice.

GCD ranges from below 1 to above 100 (Table 13) that classifies ground's hydraulic conductivity into 6 categories; very low (VL), low (L), moderate (M^-), medium (M^+), high (H), and very high (VH). It also classifies solidification quality into 6 categories as excellent (E), very good (VG), good (G), fair (F), poor (P), and very poor (VP).

Conduction of GCD test does not contain complicated procedure; however, there are some important notes that needs to be considered in measurements as follows:

- It is recommended to repeat the test for 3 times and then making an average of values to obtain a better precision and accuracy in GCD estimation.

- If during the water injection, pressure is not obtained, or it is lesser than 0.20 MPa, then the ground hydraulic conductivity would be considered as VH, which means that quality of grouting or injection executed at the location of the hole is classified as VP (Table 13). In this case the section should be further grouted/injected by a proper consolidation material/s and/or with a better configuration to obtain the targeted GCD value that designated in particular design.

- If the pressure is rapidly raised and exceeded 1 MPa, then the ground hydraulic conductivity would be considered as VL, which means that quality of grouting or injection executed at the location of the hole is classified as E (Table 13).

Appendix 2: ViD

Vibration-induced Damage (ViD) or in other words blast-induced damage includes deterioration of ground, further development of plastic zone beyond the excavation line, aggravation of overbreak, and damage to vicinity structure/s induced by vibration (Bineshian, 2021a). Peak Particle Velocity (PPV) is a suitable vibration parameter that is used to assess the ViD that is measured, calculated, or predicted using seismographs, mathematical formulas, or empirical equations respectively. In absence of measurement or calculation, empirical equations are developed. One of the first and most credible empirical PPV predictor (Eq 19) is proposed by USBM (Duvall and Fogelson, 1962).

$$\mathrm{PPV} = c_1 \left(\frac{\mathrm{Dist}}{\mathrm{CPD}_{\max}^{0.5}}\right)^{c_2} \tag{19}$$

where;

c_1	Site constant; determined by regression analysis on (Dist, PPV) or empirical values to be used
c_2	Site constant; determined by regression analysis on (Dist, PPV) or empirical values to be used
$\mathrm{CPD}_{\max}$	Maximum charge per delay (kg)
Dist	The distance between the blasting location and concerned structure or transducer (m)
PPV	Peak Particle Velocity (mm/sec)

"Dist" to be assumed as 20 m when Eq 19 is used for derivation of $\mathrm{ET_i}$ value from Table 8 (Section 3.7). c_1 and c_2 can be derived from regression analysis; however, use of empirical values for site constants (e.g., Table 14) assists in prediction of PPV.

Table 14. Site constants for USBM PPV Predictor (Eq 19); constants from Dyno Nobel (2010)

Ground Strength	Confinement Condition of Blast	Site Constants	
		c_1	c_2
Hard	Free Face	500	-1.6
Average	Free Face	1140	-1.6
Hard-Average	Heavily Confined	5000	-1.6

Considering that I-System includes the ETi in its equation as a dependent variable on PPV, the same is used to define an indicator for ViD as a function of the same, which is expressed in a percentage (Bineshian, 2021b). Mathematical form of this indicator is defined in Eq 20 and the classification for ViD is shown in Table 15 (Bineshian, 2021b).

$$\mathrm{Di} = (1 - \mathrm{ETi}) \times 100 \tag{20}$$

where;

Di	Damage Indicator (%); indicator for ViD to the structure in ground
ETi	Excavation Technique's Impact factor; part of I-System (Bineshian, 2019a, 2019b, 2020c)

Table 15. Damage Indicator's classification (Bineshian, 2021a, 2021b)

ET	PPV (mm/sec)*	ETi	Di Range (%)	Di-Class	ViD
ManDigg	-	1.00	0	D_i-01	Nil
ME/NonExBreak	< 2	0.99	0.1 - 1	D_i-02	Unscathed
ResiBlast	2 - 9	0.98	1.1 - 2	D_i-03	Unnoticeable
CommBlast	10 - 24	0.97	2.1 - 3	D_i-04	Negligible
IndBlast	25 - 59	0.96	3.1 - 4	D_i-05	Minor
InfraBlast	60 - 119	0.95	4.1 - 5	D_i-06	Mild
CtldBlast	120 - 449	0.90	5.1 - 10	D_i-07	Moderate
MineBlast	450 - 499	0.80	10.1 - 20	D_i-08	Major
ProdBlast	500 - 599	0.65	20.1 - 35	D_i-09	Destructive
UnCtldBlast	≥ 600	0.50	35.1 – 50 & 50^+	D_i-10	Catastrophic

* @ Dist = 20 m

Another method to assess the ViD is the Ground Conductivity Enhanced Factor (GC_{ef}), which is developed by author (Bineshian, 2021a, 2021b) using GCD (Bineshian, 2020a; Appendix 1). Post-blast enhanced conductivity can be a criterion for assessment of ViD to the surrounding ground of tunnel by taking the pre-blast conductivity as a reference value. Eq 21 and Table 16 provide ViD assessment based on GC_{ef} (Bineshian, 2021b).

$$GC_{ef} = \frac{GCD_e}{GCD_p} \tag{21}$$

where;
GCD Ground Conductivity Designation (Bineshian, 2020a)
GCD_e Existing GCD; post-blast measured GCD
GCD_p Pre-existing GCD; pre-blast measured GCD
GC_{ef} Ground Conductivity Enhanced Factor

Table 16. Assessment of ViD using GC_{ef} (Bineshian, 2021a, 2021b)

GC_{ef} Range	**ViD**
$GC_{ef} = 1.00$	Nil
$1.00 < GC_{ef} \leq 1.01$	Unscathed
$1.01 < GC_{ef} \leq 1.05$	Unnoticeable
$1.05 < GC_{ef} \leq 1.10$	Negligible
$1.10 < GC_{ef} \leq 1.20$	Minor
$1.20 < GC_{ef} \leq 1.50$	Mild
$1.50 < GC_{ef} \leq 2.00$	Moderate
$2.00 < GC_{ef} \leq 5.00$	Major
$5.00 < GC_{ef} \leq 15.0$	Destructive
$GC_{ef} > 15.0$	Catastrophic

HCF (McKown, 1986) is a handy (Eq 22) but inaccurate method for assessment of ViD due to considering only the length of half-barrels for assessment. Likewise, the ViD classifications proposed by researchers based on HCF are not in details. Author to make this handy assessment more detailed, proposed a classification for ViD based on HCF (Table 17) with more details in categorisation of damage (Bineshian, 2021a, 2021b) that is matched with ViD classification presented in Tables 15 and 16.

$$HCF = \frac{\sum L_{hc}}{\sum L_{ch}} \times 100 \tag{22}$$

where;
HCF Half-Cast Factor (%)
L_{ch} Length of contour hole (m)
L_{hc} Length of half-cast (m)

Table 17. ViD assessment based on HCF (Bineshian, 2021a, 2021b)

HCF Ranges	**ViD**
$HCF = 100$	Nil
$90.0 \leq HCF < 100$	Unscathed
$80.0 \leq HCF < 90.0$	Unnoticeable
$60.0 \leq HCF < 80.0$	Negligible
$40.0 \leq HCF < 60.0$	Minor
$20.0 \leq HCF < 40.0$	Mild
$10.0 \leq HCF < 20.0$	Moderate
$5.00 \leq HCF < 10.0$	Major
$2.50 \leq HCF < 5.00$	Destructive
$0 \leq HCF < 2.50$	Catastrophic

Use of Di, GCef, or HCF is the choice of designer, engineer, or geologist.

Appendix 3: Pull Length Advisor

Implementation of suitable length for pull is crucial for stability of opening in tunnelling specially under challenging condition. I-System in its earlier edition (e.g., 2019, 2020) proposed values for pull length (PL) to be considered for each (I)-Class during excavation in underground spaces; however, in this edition an equation (Eq 23) is proposed (Bineshian, 2021b) – based on the best fit on empirical data – for calculation of PL as a function of D and (I) values, which provides a better advice for safe PL based on tunnel dimension and ground's quality or condition. The "PL" recommendation that is reflected in (I)-Class in Tables 9 and 11 for underground structures should be calculated using Eq 23. The calculated value for PL should be further assessed by engineer as per site condition for the safe and efficient advance at face. When drill and blast technique (DnB) is used for excavation, Eq 24 provides an estimate for the drilling length (DL) for an optimised engineered blasting with a reasonable blasting efficiency.

$$\mathrm{PL} = 0.5\mathrm{D}\frac{(\mathrm{I})}{100} \tag{23}$$

$$\mathrm{DL} = 1.1\mathrm{PL} \tag{24}$$

where;
(I) I-System's value
D Diameter, width, or height (mm) of underground opening (the greater value)
DL Drilling Length (mm) when DnB is used for excavation
PL Pull Length (mm) as an advice for advance in tunnel using DnB or ME

Figure 17 is a graphical representation of Eq 23. Calculated PL from Eq 23 or derived from Figure 17 can be further reviewed by the engineer at site based on the actual condition of ground and structure.

An example as a guide is also shown in Figure 17 for a tunnel with D = 8,000 mm and (I) = 75; the PL for DnB technique for tunnel's face advance is derived from Figure 17 and DL is calculated using Eq 24:

PL = 3000 mm

DL = 3300 mm

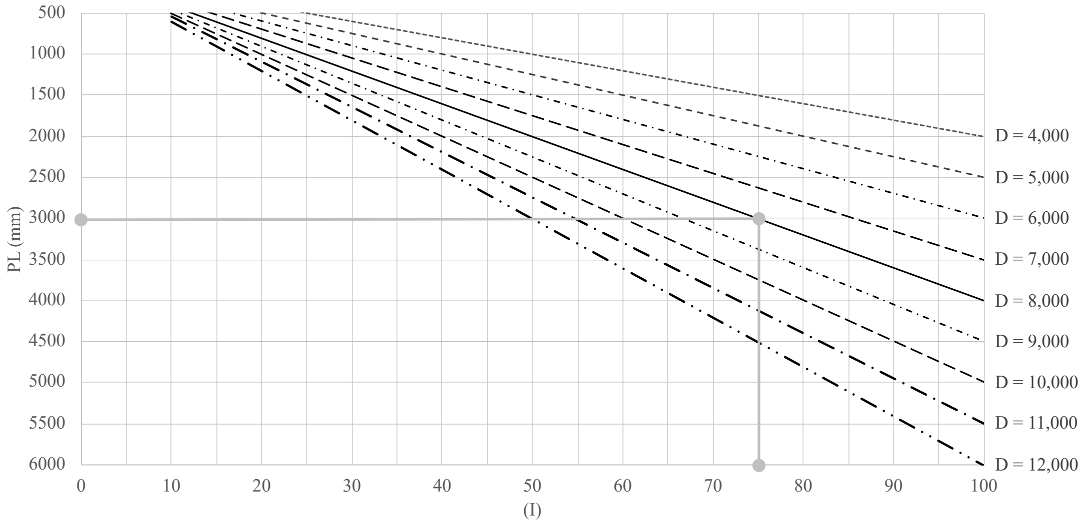


Figure 17. PL graph; derivation of PL as a function of D and (I) for tunnels (Bineshian, 2021b)

Appendix 4: SRH

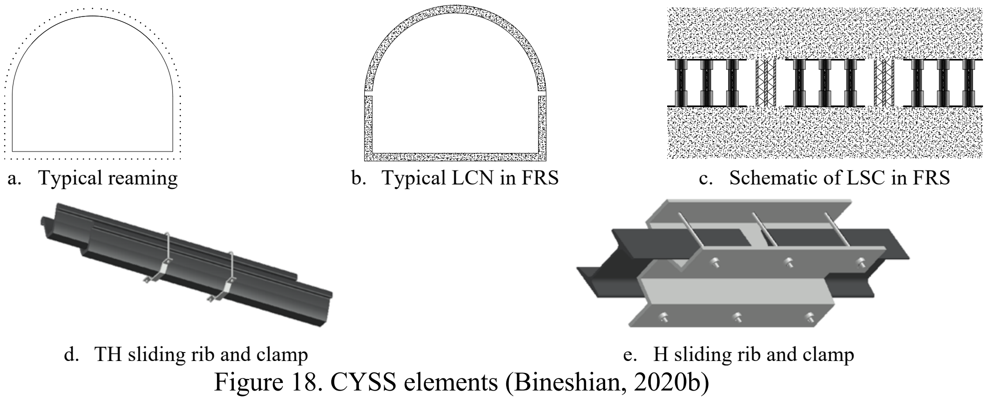
SSH (squeezing, swelling, and heaving) behaviour is a complicated non-uniform time-dependent mechanical response of ductile ground to excavation; however, it is different in load configuration compared to creep as a typical time dependent behaviour. It is a post-excavation procedure of yield stress development that generates plastic zone around the opening, however; because of stimulation caused by micro-scale sliding failure of existing non-uniformly distributed weak planes, shear stresses further magnified that initiates non-uniform deformation toward free space in a plastification process. Convergence is occurred when excavated space is the only existing free space. Severity of SSH condition depends on ground properties and induced stresses (Bineshian, 2020b). An empirical identification criterion for distinguishing SSH from Non-SSH as well as a classification for severity of SSH is presented in Table 18.

Table 18. Identification criterion and classification for SSH condition (Bineshian, 2020b)

Convergency (mm)	SSH-Class
$C \leq \frac{0.5D}{100}$	Non-SSH
$\frac{0.5D}{100} < C \leq \frac{2D}{100}$	Minor
$\frac{2D}{100} < C \leq \frac{4D}{100}$	Mild
$C > \frac{4D}{100}$	Severe

C Convergency (mm)
D Diameter, width, or height (mm) of underground opening (the greater value)

Support system used for SSH ground includes Rigid Support System (RSS), and Yield/Ductile Support System (YSS). RSS is a stiff system of measures to resist against induced deformation due to SSH behaviour by absorbing entire accumulative SSH stresses. RSS is applicable for passive load configuration in gravity driven and shallow depth overburden with less arch effect; it is not recommended to be applied for tunnelling in SSH ground. YSS is designed to accommodate deformations that is induced to periphery of tunnel in SSH ground by controlled yielding to prevent/terminate accumulation of load. Application of YSS in its conventional form (CYSS) includes reaming (Figure 18a), LCN (Figure 18b), LSC (Figure 18c), YieldR (Figures 18d and 18e), YieldB, convergency measurements, and instrumentation.



a. Typical reaming b. Typical LCN in FRS c. Schematic of LSC in FRS
d. TH sliding rib and clamp e. H sliding rib and clamp

Figure 18. CYSS elements (Bineshian, 2020b)

Application of CYSS is a cost- and time-inefficient system, which includes several sequences as well as delay and hindrance in the tunnelling procedures. Consequently, SRH (Stress Release Hole/s) is developed for tunnelling under SSH condition (Bineshian, 2020b) to eliminate the hazards and challenges involved with tunnelling under SSH condition as well as lessening the hindrance and cost caused by application of CYSS without compromise in safety. Key concept behind SRH is to divert the SSH stresses towards the uniformly distributed free spaces created by SRHs; therefore, non-uniform deformation caused by SSH behaviour is induced to the SRHs instead of their occurrence on periphery (Figure 19). Thus, further accumulation of SSH stresses also will be controlled and finally terminated. Table 19 offers requirements for application of SRH System for treatment of each class of SSH condition in tunnelling as well as requirements for CYSS. SRH System releases/terminates incremental non-uniform time-dependent shear stresses around the periphery, prevents/minimises convergency, eliminates repair or rework of primary SS, saves in cost and time and reduces hindrance compared to CYSS, and also improves efficiency of advancement (Bineshian, 2020b).

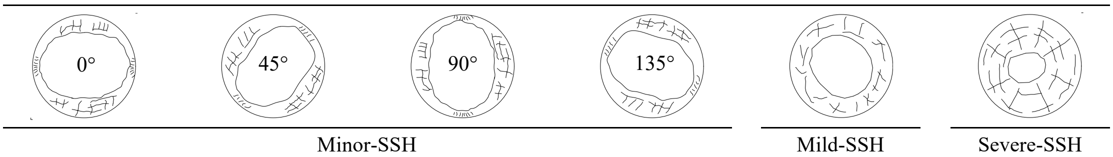


Figure 19. Illustration of observed patterns of induced deformation inside SRH (Bineshian, 2020b)

Table 19. Guideline for requirements in application of SRH and/or CYSS (Bineshian, 2020b)

Main Required Resources	**SSH-Class**					
	Minor*		Mild*		Severe*	
	CYSS	SRH	CYSS	SRH	CYSS	SRH
Reaming	n/a	n/a	a	n/a	a	a
YieldR ^	a	n/a	a	n/a	a	a
LG (If TH or H profile is not used)	a	n/a	n/a	c/a	n/a	c/a
RingC<	n/a	n/a	a	n/a	a	c/a
FRS	a	a	a	a	a	a
LCN>	a	n/a	a	n/a	a	a
LSC and laxation of clamps>	a	n/a	a	n/a	a	n/a
YieldB using SysDB (L = 0.5D)	n/a	n/a	a	n/a	a	a
YieldB using SN, SDA, Swellex (L = 0.5D)	a	n/a	n/a	a	n/a	n/a
Drilling of 100 - 300 mm holes for SRH (L = 1D)	n/a	a	n/a	a	n/a	a
3DM or Chord Convergency Meter@	a	a	a	a	a	a
DIC# @ 25 m	n/a	n/a	a	a	n/a	n/a
Strain Meter @ 100 m	a	n/a	a	n/a	a	a
Pressure Cell or Load Cell @ 150 m	a	n/a	a	n/a	a	a
Single-/Multi-Rod Extensometer @ 300 m	n/a	n/a	a	n/a	a	a
Strain Gauge @ 400 m	n/a	n/a	n/a	n/a	a	a

*	Table 18.
^	TH or H profile capable of sliding; Figures 18d and 18e.
<	Ring Closure or Invert Closure; It is recommended to be applied to prevent heaving; however, its applicability depends on observations, SSH-Class, and (I)-Class to be decided by Engineer at site.
>	Figure17b; skilled team for installation and deformation control is required.
#	Bineshian et al (2021a, 2021b)
@	Spacing between the measuring stations (Table 20)
a	Applicable
c/a	Conditionally Applicable; applicable for (I)-07; not applicable for (I)-05 and (I)-06
D	Diameter, width, or height (mm) of underground opening (the greater value)
L	Length (mm)
n/a	Not Applicable

SRH System is applied in a systematic pattern of large diameter drilled – using ordinary rotary percussion drilling system – holes (100 - 300 mm) in an individual system as shown in Figure 20 or combined with other measures (Table 19) depending on the severity of convergency. Continues monitoring of stresses is required when CYSS (e.g., sliding ribs and LSC) is applied, further to several periodical sequences of measurements and clamps laxation, which is time-consuming procedure that hinders the advancement. Contrastingly, SRH replaces all these sequences of CYSS with a simple systematic inexpensive drilling of holes. SRH works in a continuous no-maintenance manner until it ends the accumulation of shear stresses and convergency. It is individually applicable for Minor to Mild-SSH condition with (I)-05 to (I)-07 without needs of CYSS. Moreover, it is applicable for Severe-SSH in combination with some elements of CYSS (Table 19). When LG is applied in combination with SRH in ground with (I)-07, it prevents occurrence of large deformation at periphery while SRH System absorbs the deformation beyond the periphery (Bineshian, 2020b).

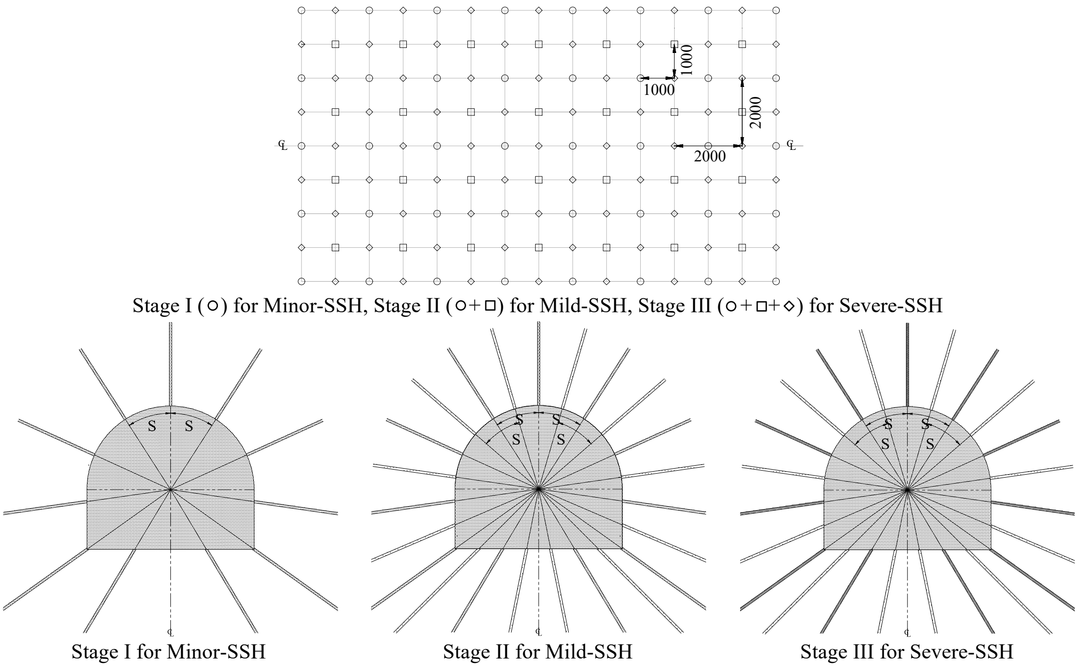


Figure 20. SRH drilling pattern – plan and cross section views; L = D, S = 2000 mm (Bineshian, 2020b)

3DM measures for tunnelling under SSH condition is provided in Table 20. Each 3DMS may contain 3, 5, or 7 BRTs depending on the SSH-Class (Figure 21) and the size of the underground space. It is not recommended to place the final liner before termination of convergency and earlier than ending of proposed minimum period of monitoring.

Table 20. 3DM; guideline for application in tunnelling under SSH condition (Bineshian, 2020b)

SSH-Class	Number of BRTs at each 3DMS	Frequency of Reading	Minimum period of monitoring (month)	Spacing of 3DMS (m)
Non-SSH	Measures proposed at (I)-Class in I-System (Tables 9 – 12 in Section 4) is applicable; (I)-01 to (I)-10			
Minor	3	Once a fortnight	6	15
Mild	5	Once a week	9	10
Severe	7	Twice a week	12	5

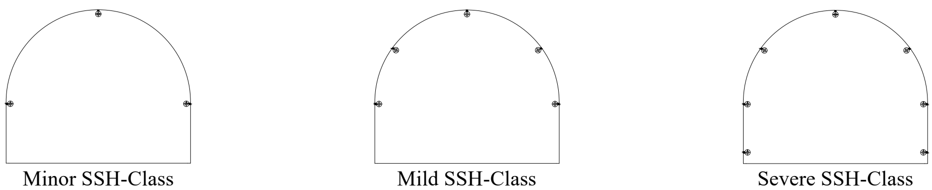
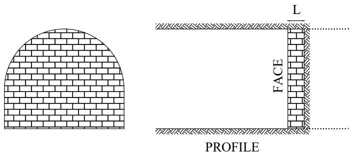


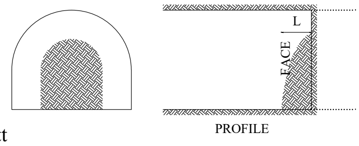
Figure 21. 3DM; illustration of configuration of 3DMS based on SSH-Class (Bineshian, 2020b)

Similarly, SRH System is applicable for tunnelling in burst prone (BP) condition; however, if TBM is used application of McNally et al (2002) support system with HEAM/WeldM is an alternative choice. Use of SRH System to control the plastic zone around the tunnel in BP load configuration is recommended with use of CtldBlast (PPV ≤ 449 mm/sec), SysDB, FRS, and HEAM or WeldM (Bineshian, 2020b).

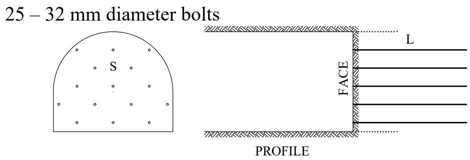
Appendix 5: Schematic Illustrations of SS



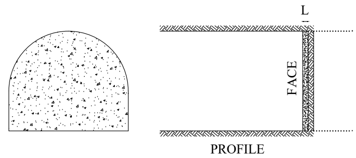
a. BulkH
L ≤ 0.15D



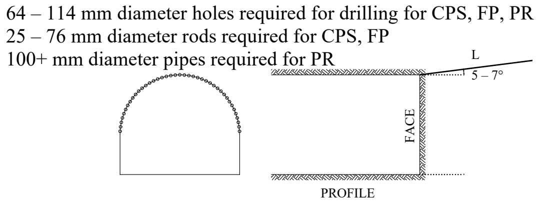
b. FaceButt
L ≥ 0.25D



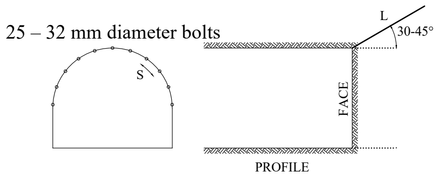
c. FaceB, FibreD
L = D and S ≤ 0.3L



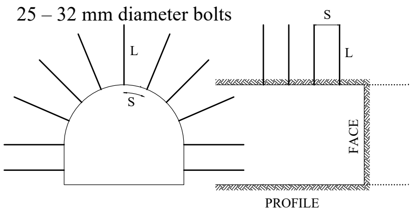
d. FaceP
L ≤ 0.05D



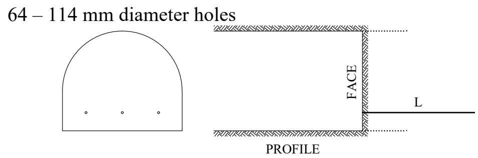
e. CPS, FP, PR
L = 0.7D to L = D



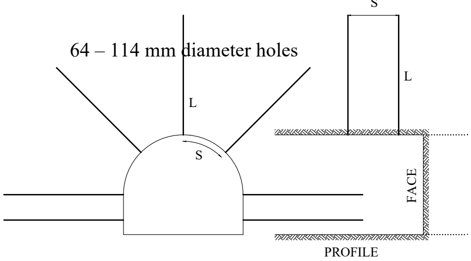
f. SysHB
L = 0.7D; S ≤ 0.3L



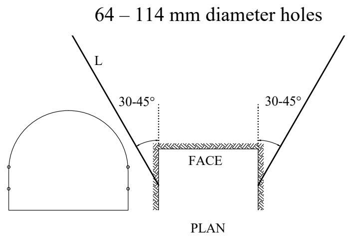
g. ConeB, SysB, SysDB, YieldB; L = 0.5D
SysLB; L = 0.7D, SysN; L= D; S = 0.3L



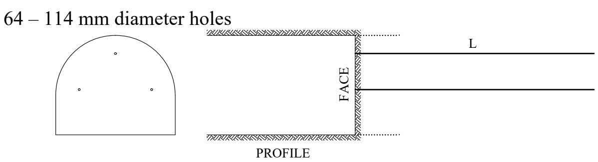
h. ADH
L ≤ 1.5D



i. RDH
L ≤ D



j. WDH
L ≤ 2D



k. BH, EC, PH
L = 2D to L = 3D

Figure 22. Schematical illustration of some SS elements proposed in (I)-Class; Sections 4 and 10

Appendix 6: Systematic Bolting Calculator

Estimation of the length of bolting system (Section 10) is summarised here in Table 21, which is proposed for (I) ≤ 50 (Tables 9 and 11) to provide an initial estimation for length and spacing of bolts as variables dependent on D; however, due to having only one independent variable without considering the ground quality, it gives less precision and accuracy in estimation of bolt parameters when ground condition varies.

Table 21. Estimation of the length of systematic bolting proposed in I-System for underground works

Bolting Parameters	Systematic Bolting*					
	ConeB	SysB	SysDB	YieldB	SysLB	SysN
L	0.5D				0.7D	D
S	0.3L					

* Using 25-32 mm diameter steel-bar/-pole including (e.g., SDA, SN, etc.)
D Diameter, width, or height (mm) of underground opening (the greater value)
L Length of ConeB, SysB, SysDB, SysLB, SysN, YieldB (mm)
S Spacing of bolts along the both axis and transverse direction (mm)

Author proposed Eq 25 and 26 for calculation of length and spacing of aforementioned bolting systems as a function of D and (I). Eq 25 is valid for (I) ≤ 50. Systematic bolting for (I) > 50 is not recommended in I-System; instead, spot and/or individual bolting is recommended (if required). Neither to be conservative nor incautious, a comparison of output of Table 21 and Eq 25 and 26 may help to decide on bolting system's parameters.

$$L = D \times \frac{100-(I)}{100} \tag{25}$$

$$S = 0.3L \tag{26}$$

where;
(I) I-System's value
D Diameter, width, or height (mm) of underground opening (the greater value)
L Length of ConeB, SysB, SysDB, SysLB, SysN, YieldB (mm) for 25-32 mm diameter bolts
S Spacing of bolts along the both axis and transverse direction (mm)

Figure 23 provides a graph based on Eq 25 for L when D and (I) are known. An example is also shown in Figure 23 for a tunnel with D = 10,000 mm and (I) = 30; the length of the bolting system is derived as L = 7000 mm.

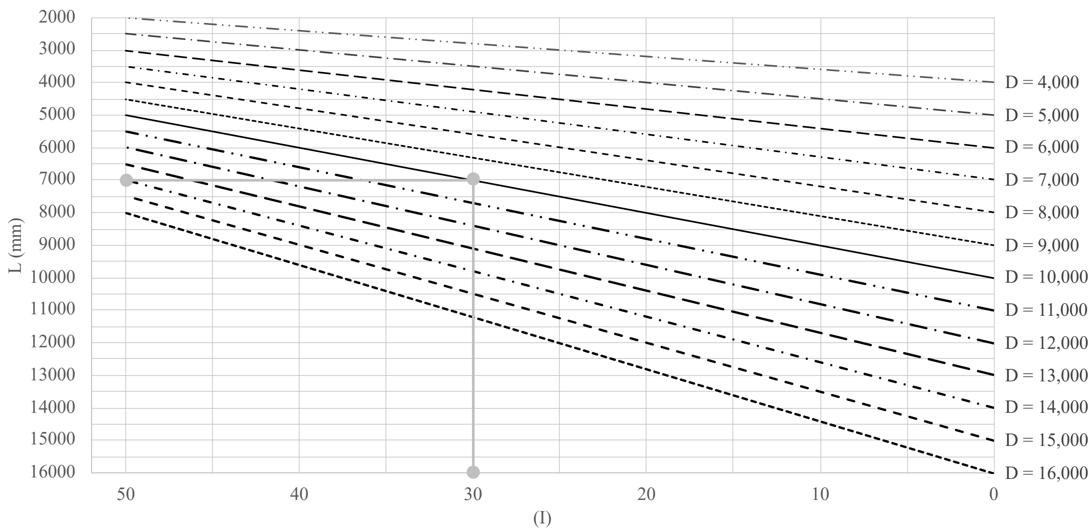


Figure 23. L graph; derivation of length of bolting systems (25-32 mm diameter ConeB, SysB, SysDB, SysLB, SysN, YieldB) as a function of D and (I) for underground spaces

Appendix 7: I-System Software – Input and Output

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi — Bineshian (2019)

Calculation Sheet: CH46598	Location: JK
Project: USBRL-T05	Type of Structure: Underground
Country: IN	Date: 2021/03/11

Ai - ARMATURE INDEX: 2.77

dn Discontinuity Number/s - per m	≥ 25
ds Discontinuity Set/s	3
di Discontinuity Inclination - °	31 - 60
da Discontinuity Aperture	Open
dd Discontinuity Disintegration	Semi-Integrated
df Discontinuity Friction	Low Friction - Smooth/Even
dp Discontinuity Persistency	≥ 0.90 x D

Ci - CONFIGURATION INDEX: 5.25

pc Problematical Configuration	Sheared - High Shear Stresses - e.g. Mylonite
sc Structural Configuration	Layered (100 - 10 cm)

Hi - HYDRO INDEX: 6.50

gc Ground Conductivity (GCD) || [Wetness]	(7 - 9.99) || [Wet]
gs Ground Softness - Mohs	5

Pi - PROPERTIES INDEX: 6.60

cc Cohesiveness Consistency	Picked Easily
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	Sand
pm Particle Morphology	Sub-angular
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(3499 - 3000) || [1999 - 1500]

Si - STRENGTH INDEX: 8.10

cs UCS	19 - 10 MPa
se Scale Effect	D/H = 1.20 - 0.80 & σv ≥ σh

DFi - DYNAMIC FORCES IMPACT: 0.85

(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}

ETi - EXCAVATION TECHNIQUE IMPACT: 0.99

(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

Figure 24a. I-System Software's output; Input

I-System - Index of Ground-Structure Bineshian (2019)

25%

(I)-Class

(I)-08

Recommended Measure/s

SS - Support System

FP32.200.L.X1/FP76.250.L.X1/PR100.300.L.X1, SysLB32.L.S, LG32.25.180.1000-/RigidR150UC23.1000-, FRS225/FRC225, FaceButt.L, FRFS200, RDH54.L+CF

ET - Excavation Technique/s

PSE-ME/NonExBreak, PL1000-

IT - Instrumentation Technique/s

3DMS@50m, StrainM@200m, PressC/LoadC@250m, SingleRodE@400m

PT - Prevention Technique/s

Apply FP/PR, Maintain Buttress, Avoid: FF & DnB

FT - Forecast Technique/s

TSP/PH54.EC.L

Design Remark/s

Passive load configuration, sensitive to scale, unsupported span, & stand-up time

Figure 24b. I-System Software's output; (I)-Class Output

I-System - Index of Ground-Structure Bineshian (2019)

(I)-GC; I-System's Ground Characterization

(I) = 25

Selected UCS range is 19 - 10 MPa.

Specified σ_c Value = 10 MPa

Modulus of Deformation

E_g = 2.490 GPa

Poisson's Ratio

ν_g = 0.400

Unconfined Compressive Strength

σ_{cg} = 0.244 MPa

Uniaxial Tensile Strength

σ_{tg} = -0.012 MPa

Cohesion

C_g = 1.706 KPa

Internal Friction Angle

φ_g = 28.750°

(I)-GC characterizes the ground based on (I); however, it is recommended to scrutinise it by deriving the mechanical properties of ground by standardised in-situ testing methods.

Figure 24c. I-System Software's output; (I)-GC Output

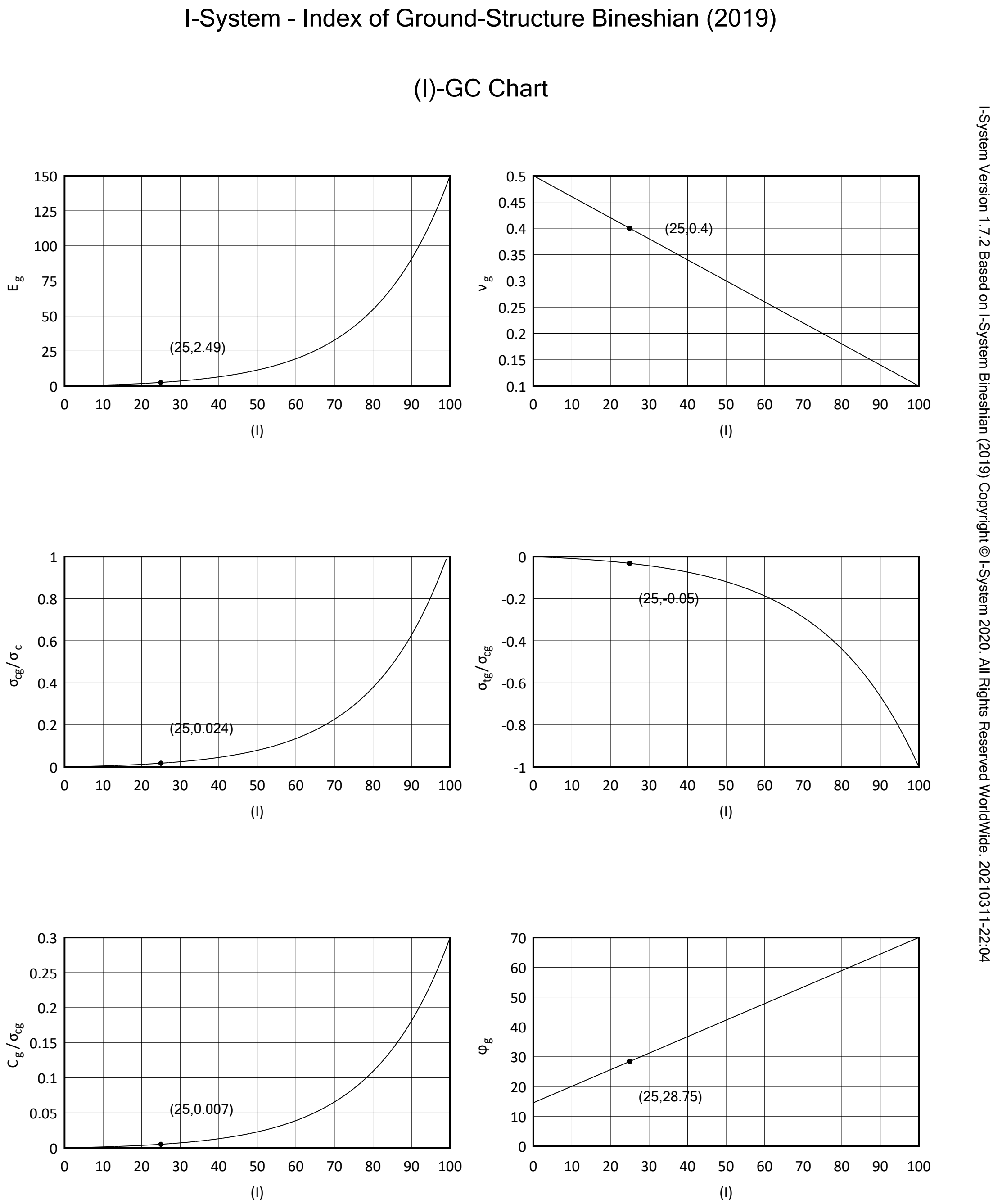


Figure 24d. I-System Software's output; (I)-GC Chart Output

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Ground Characterisation for Design using I-System Software

Case Studies

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Abstract

This article aims to provide case studies of recent use of I-System in underground, semi-surface, and surface projects in both rocks and soils. I-System is used for classification and characterisation of ground in the projects in India including USBRL and Teesta in design and/or for execution of the work. Recommendations obtained form (I)-Class including primary and final support system, required excavation technique/s for encountered condition, proper instrumentation technique/s for monitoring, appropriate prevention technique/s against possible failures, verified forecast technique/s to predict the ground condition ahead, and practical design recommendations for understanding of ground behaviour, failure mechanism, and load configuration ascertained to be practical and well optimised. Characterisation of ground's mechanical properties by (I)-GC including modulus of deformation, Poisson's ratio, unconfined compressive strength, uniaxial tensile strength, cohesion, and internal friction angle are found to be suitable to be used in design.

Keywords: (I)-Class, (I)-GC, I-System

Nomenclature

(I)	I-System's value
(I)-Class	I-System's Ground Classification
(I)-GC	I-System's Ground Characterisation
Ai	Armature Index
C_g	Cohesion of ground
Ci	Configuration Index
D	Diameter/width/height of opening (the greater value)
DFi	Dynamic Forces Impact
E_g	Deformation Modulus of ground
ET	Excavation Technique/s
ETi	Excavation Technique Impact
ERZ	Earthquake Risk Zone
FT	Forecast Technique/s
Hi	Hydro Index
IT	Instrumentation Technique/s
KRCL	Konkan Railway Corporation Limited
LTHPL	Lanco Teesta Hydro Power Limited
MSK	Medvedev-Sponheuer-Karnik Scale
NATM	New Austrian Tunnelling Method
NHPC	National Hydroelectric Power Corporation
NR	Northern Railway of Indian Railway
PGA_{SD}	Scaled Design Peak Ground Acceleration (g)
Pi	Properties Index
PPV	Peak Particle Velocity
PT	Prevention Technique/s
Si	Strength Index
SS	Support System
UCS	Uniaxial Compressive Strength
USBRL	Udhampur-Srinagar-Baramula Rail Link
ν_g	Poisson's Ratio of ground
σ_{cg}	Unconfined Compressive Strength of ground
σ_{tg}	Uniaxial Tensile Strength of ground
φ_g	Internal Friction Angle of ground

1. Introduction

I-System (Bineshian, 2019a, 2019b, 2020) has been verified in railway, metro, road, canal, hydropower, and mining projects during 22 years course of development. After it was first published in 2019, it is further applied in design of several projects in India including Tunnels T03, T05, T06, T35, T37, T44, T50, and T51 of Central Railway, Tunnels T05 wider section, T09 cut and cover, T40 wider section, T41-T47, and T14 twin tunnels of Northern Railway, Tunnels Pernem and Old Goa of Konkan Railway, and Teesta hydropower project. The results were quite satisfactory. Following sections provide a brief study on application of I-System in some of mentioned projects of recent cases. Calculation procedure is explained in details in I-System paper published in 2019 (Bineshian, 2019b); therefore, it is avoided here to repeat the calculation procedure.

2. Underground Rock Works – USBRL Project – Tunnel T05

NR is the client of the project. Work is being executed by KRCL. T05 is one of the most challenging NATM tunnels in USBRL in North India (Bineshian, 2017a, Bineshian et al, 2019). It is a 6 km twin tube tunnel in young Himalaya's terrain in dolomite rock formation, which passes through several hazardous zones. Following case is related to an incident (2019/05/11) at main tunnel (D = 8000 mm) at CH46598. I-System is utilised to assess the condition and to provide a solution to overcome the challenge. I-System's recommendations are applied and successful results obtained.

Figure 1 represents the input and output data of analysis obtained from I-System Software.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi Bineshian (2019)

Calculation Sheet: CH46598	Location: JK
Project: USBRL-T05	Type of Structure: Underground
Country: IN	Date: 2021/03/11

Ai - ARMATURE INDEX: 2.77

dn Discontinuity Number/s - per m	≥ 25
ds Discontinuity Set/s	3
di Discontinuity Inclination - °	31 - 60
da Discontinuity Aperture	Open
dd Discontinuity Disintegration	Semi-Integrated
df Discontinuity Friction	Low Friction - Smooth/Even
dp Discontinuity Persistency	≥ 0.90 x D

Ci - CONFIGURATION INDEX: 5.25

pc Problematical Configuration	Sheared - High Shear Stresses - e.g. Mylonite
sc Structural Configuration	Layered (100 - 10 cm)

Hi - HYDRO INDEX: 6.50

gc Ground Conductivity (GCD) || [Wetness]	(7 - 9.99) || [Wet]
gs Ground Softness - Mohs	5

Pi - PROPERTIES INDEX: 6.60

cc Cohesiveness Consistency	Picked Easily
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	Sand
pm Particle Morphology	Sub-angular
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(3499 - 3000) || [1999 - 1500]

Si - STRENGTH INDEX: 8.10

cs UCS	19 - 10 MPa
se Scale Effect	D/H = 1.20 - 0.80 & σv ≥ σh

DFi - DYNAMIC FORCES IMPACT: 0.85

(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}

ETi - EXCAVATION TECHNIQUE IMPACT: 0.99

(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

Figure 1a. I-System Software output – USBRL – T05 Tunnel

I-System - Index of Ground-Structure Bineshian (2019)

25%

(I)-Class

(I)-08

Recommended Measure/s

SS - Support System

FP32.200.L.X1/FP76.250.L.X1/PR100.300.L.X1, SysLB32.L.S, LG32.25.180.1000-/RigidR150UC23.1000-, FRS225/FRC225, FaceButt.L, FRFS200, RDH54.L+CF

ET - Excavation Technique/s

PSE-ME/NonExBreak, PL1000-

IT - Instrumentation Technique/s

3DMS@50m, StrainM@200m, PressC/LoadC@250m, SingleRodE@400m

PT - Prevention Technique/s

Apply FP/PR, Maintain Buttress, Avoid: FF & DnB

FT - Forecast Technique/s

TSP/PH54.EC.L

Design Remark/s

Passive load configuration, sensitive to scale, unsupported span, & stand-up time

Figure 1b. I-System Software output – USBRL – T05 Tunnel

I-System - Index of Ground-Structure Bineshian (2019)

(I)-GC; I-System's Ground Characterization

(I) = 25

Selected UCS range is 19 - 10 MPa.

Specified σ_c Value = 10 MPa

Modulus of Deformation

E_g = 2.490 GPa

Poisson's Ratio

ν_g = 0.400

Unconfined Compressive Strength

σ_{cg} = 0.244 MPa

Uniaxial Tensile Strength

σ_{tg} = -0.012 MPa

Cohesion

C_g = 1.706 KPa

Internal Friction Angle

φ_g = 28.750°

(I)-GC characterizes the ground based on (I); however, it is recommended to scrutinise it by deriving the mechanical properties of ground by standardised in-situ testing methods.

Figure 1c. I-System Software output – USBRL – T05 Tunnel

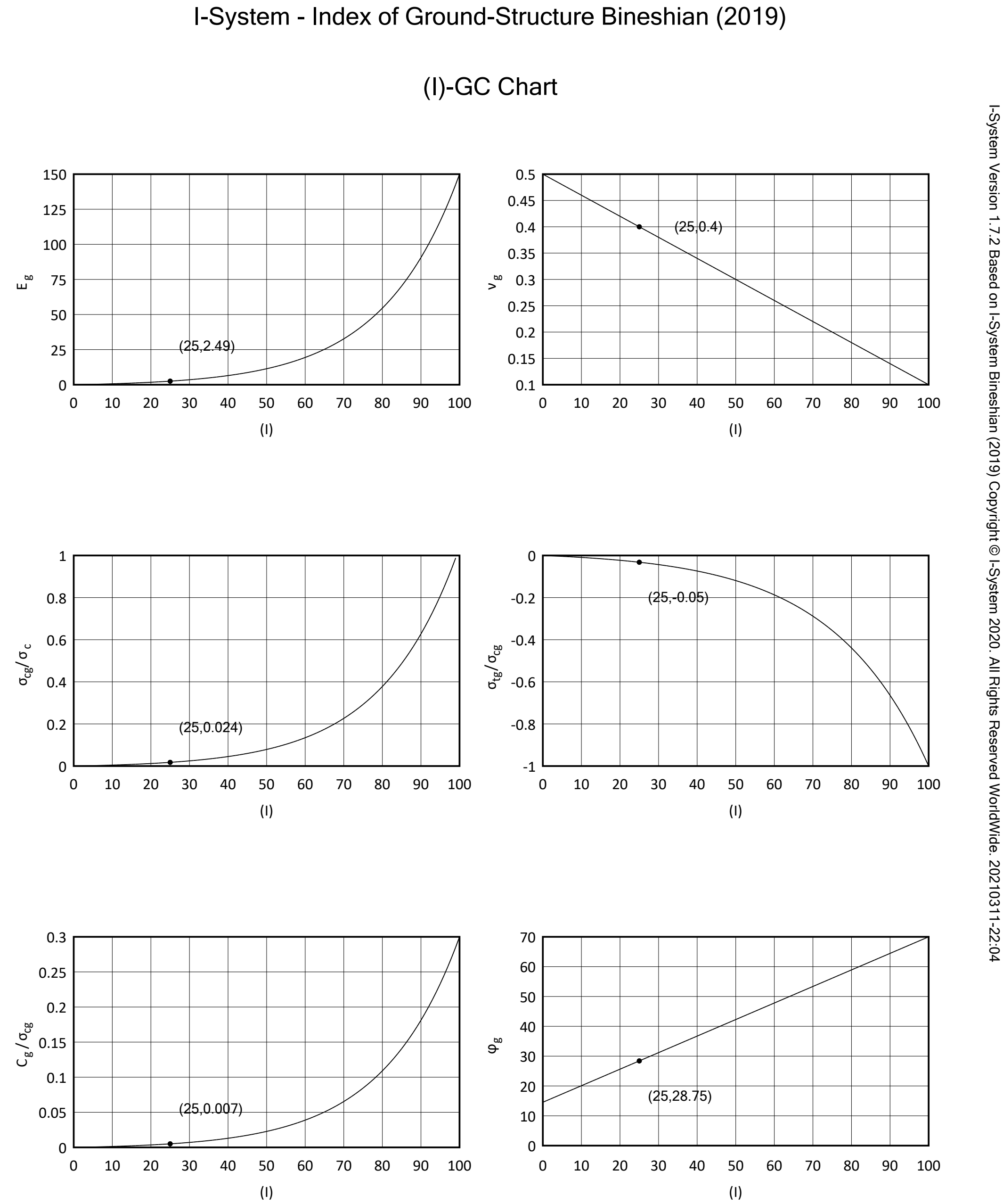

Figure 1d. I-System Software output – USBRL – T05 Tunnel

3. Underground Soil Works – USBRL Project – Tunnel T02

Tunnel T02 is another challenging NATM tunnel (D = 8000 mm) in USBRL (Client: NR, Engineer in charge: KRCL) with a length of almost 5.6 km twin tube in young Himalaya's terrain (Bineshian, 2017b). For some stretches, it passes through extremely challenging soil formation. Advancing at face of the main tunnel at CH37488 was impossible on 2018/09/23 due to gravity driven behaviour of ground. Condition is assessed by I-System. Work is resumed after application of I-System's classification recommendations and characterisation's design parameters; no instabilities occurred in the course of tunnelling in the zone.

Figure 2 shows the input and output of analysis using I-System Software.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi

Bineshian (2019)

Calculation Sheet:	Failure	Location:	JnK
Project:	USBRL - T02	Type of Structure:	Underground
Country:	IN	Date:	2021/06/03

Ai - ARMATURE INDEX: 0.00

dn	Discontinuity Number/s - per m	N/A || Jointless
ds	Discontinuity Set/s	N/A || Jointless
di	Discontinuity Inclination - °	N/A || Jointless
da	Discontinuity Aperture	N/A || Jointless
dd	Discontinuity Disintegration	N/A || Jointless
df	Discontinuity Friction	N/A || Jointless
dp	Discontinuity Persistency	N/A || Jointless

Ci - CONFIGURATION INDEX: 4.00

pc	Problematical Configuration	Homogeneous || Isotropic || Jointless || Granular
sc	Structural Configuration	Cohesive Matrix Skeleton

Hi - HYDRO INDEX: 2.40

gc	Ground Conductivity (GCD) || [Wetness]	(25 - 49) || [Flow]
gs	Ground Softness - Mohs	4

Pi - PROPERTIES INDEX: 8.12

cc	Cohesiveness Consistency	Picked Easily
dc	Denseness Consistency	Never Indented by Thumbnail
ps	Particle Size	Gravel
pm	Particle Morphology	Angular
bw	Body Wave Velocity - m/sec (Vp) || [Vs]	(3999 - 3500) || [2199 - 2000]

Si - STRENGTH INDEX: 7.20

cs	UCS	9 - 5 MPa
se	Scale Effect	D/H = 1.20 - 0.80 & σv ≥ σh

DFi - DYNAMIC FORCES IMPACT: 0.85

(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}

ETi - EXCAVATION TECHNIQUE IMPACT: 0.90

(ET) || [PPV mm/sec]	(CtldBlast) || [120 - 449]

Figure 2a. I-System Software output – USBRL – T02 Tunnel

I-System - Index of Ground-Structure Bineshian (2019)

17%

(I)-Class

(I)-09

Recommended Measure/s

SS - Support System

PR100.250.L.X1/FP76.200.L.X1/FP32.200.L.X2, FaceB25.L.S/FaceP300-, FaceButt.L, PreG/I, RigidR150UC23.750-+RingC, SysN32.L.S, FRS225/FRC225, FRFS200, RDH54.L+CF

ET - Excavation Technique/s

PSD-ME, PL750-

IT - Instrumentation Technique/s

3DMS@25m, StrainM@150m, PressC/LoadC@200m, MultiRodE@400m, StrainG@500m

PT - Prevention Technique/s

Apply PreG/I & PR/FP, Maintain Buttress, Avoid: FF, NonExBreak/DnB, & Ductile SS

FT - Forecast Technique/s

TSP/PH54.EC.L

Design Remark/s

Passive load configuration, sensitive to scale, unsupported span, & stand-up time

Figure 2b. I-System Software output – USBRL – T02 Tunnel

I-System - Index of Ground-Structure Bineshian (2019)

(I)-GC; I-System's Ground Characterization

(I) = 17

Selected UCS range is 9 - 5 MPa.

Specified σ_c Value = 5 MPa

Modulus of Deformation

E_g = 1.340 GPa

Poisson's Ratio

ν_g = 0.432

Unconfined Compressive Strength

σ_{cg} = 0.082 MPa

Uniaxial Tensile Strength

σ_{tg} = -0.003 MPa

Cohesion

C_g = 0.383 KPa

Internal Friction Angle

φ_g = 24.350°

(I)-GC characterizes the ground based on (I); however, it is recommended to scrutinise it by deriving the mechanical properties of ground by standardised in-situ testing methods.

Figure 2c. I-System Software output – USBRL – T02 Tunnel

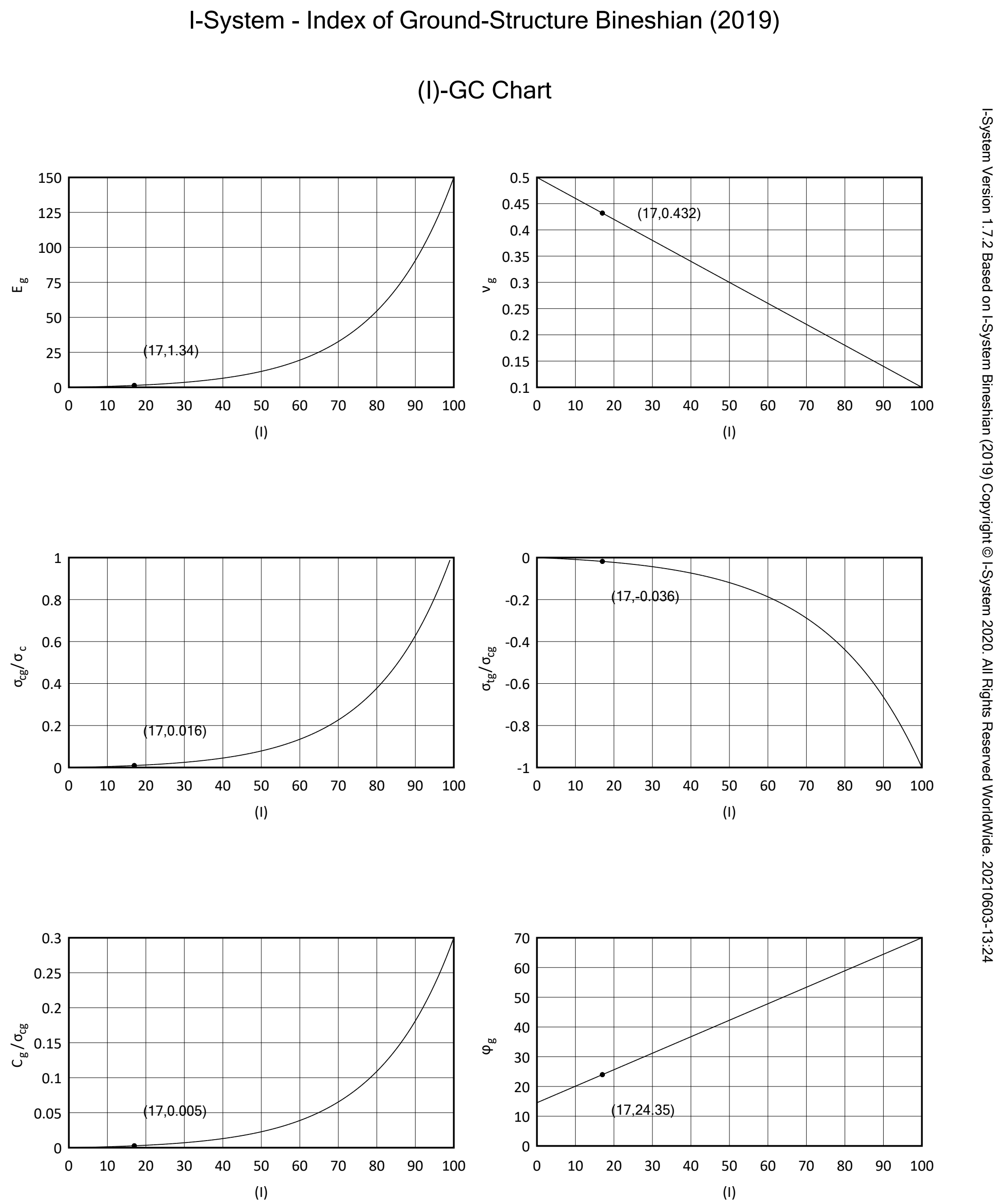

Figure 2d. I-System Software output – USBRL – T02 Tunnel

4. Surface Rock and Soil Works – USBRL Project – Tunnel Portal of T13

Portal works of both main and escape tunnels of T13 (one of the longest twin tunnels in USBRL with 9 km length in young Himalaya's terrain) is designed using I-System. Following case represents the output for I-System evaluation on portal of the escape tunnel of T13. The slope of this portal is located in a ground with mix of sandstone fragments in soil matrix. (I)-Class output obtained from I-System for the slope of the T13's portal is being applied. The portal is designed using (I)-GC output of I-System as design parameters.

Input as well as output of the analysis conducted using I-System Software is presented in Figure 3.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi Bineshian (2019)

Calculation Sheet: ET Portal	Location: JnK
Project: USBRL - T13	Type of Structure: Surface
Country: IN	Date: 2021/06/03

Ai - ARMATURE INDEX: 5.48

dn Discontinuity Number/s - per m	15 - 19
ds Discontinuity Set/s	3
di Discontinuity Inclination - °	11 - 30
da Discontinuity Aperture	Semi-Tight
dd Discontinuity Disintegration	Weathered/Altered
df Discontinuity Friction	Moderate Friction - Nonsmooth
dp Discontinuity Persistency	≥ 0.90 x D

Ci - CONFIGURATION INDEX: 9.00

pc Problematical Configuration	Fractured - Highly
sc Structural Configuration	Layered (100 - 10 cm)

Hi - HYDRO INDEX: 12.00

gc Ground Conductivity (GCD) || [Wetness]	(≤ 0.99) || [Dry]
gs Ground Softness - Mohs	6

Pi - PROPERTIES INDEX: 16.00

cc Cohesiveness Consistency	Indurated
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	N/A (e.g. Grainless)
pm Particle Morphology	N/A (e.g. Grainless)
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(4999 - 4500) || [2899 - 2600]

Si - STRENGTH INDEX: 12.60

cs UCS	74 - 50 MPa
se Scale Effect	B/H = 1.20 - 0.80

DFi - DYNAMIC FORCES IMPACT: 0.85

(PGASD) || [ERZ] || {MSK}	(0.36g - 0.50g) || [VH] || {IX-X}

ETi - EXCAVATION TECHNIQUE IMPACT: 0.99

(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

Figure 3a. I-System Software output – USBRL – T13 Tunnel

I-System - Index of Ground-Structure Bineshian (2019)

46%

(I)-Class

(I)-06

Recommended Measure/s

SS - Support System

Scng, SysA25.L.S, FRS150, DH54.L

ET - Excavation Technique/s

(PreS, DD6000-), (ProdBlast, PD2000-)

IT - Instrumentation Technique/s

3DMS@75m, IncM@500m

PT - Prevention Technique/s

Cover Slope Crest with WPM & FRS at a Width Equal to Height to Help Prevention of Tension Crack Generation, Avoid: ProdBlast/UnCtldBlast, Surcharge at Crest, & Toe Lightening

FT - Forecast Technique/s

ERT/VPH54.L

Design Remark/s

Check against plain/wedge/toppling failure & rock fall criteria

Figure 3b. I-System Software output – USBRL – T13 Tunnel

I-System - Index of Ground-Structure Bineshian (2019)

(I)-GC; I-System's Ground Characterization

(I) = 46

Selected UCS range is 74 - 50 MPa.

Specified σ_c Value = 50 MPa

Modulus of Deformation

E_g = 8.974 GPa

Poisson's Ratio

ν_g = 0.316

Unconfined Compressive Strength

σ_{cg} = 3.491 MPa

Uniaxial Tensile Strength

σ_{tg} = -0.403 MPa

Cohesion

C_g = 69.639 KPa

Internal Friction Angle

φ_g = 40.300°

(I)-GC characterizes the ground based on (I); however, it is recommended to scrutinise it by deriving the mechanical properties of ground by standardised in-situ testing methods.

Figure 3c. I-System Software output – USBRL – T13 Tunnel

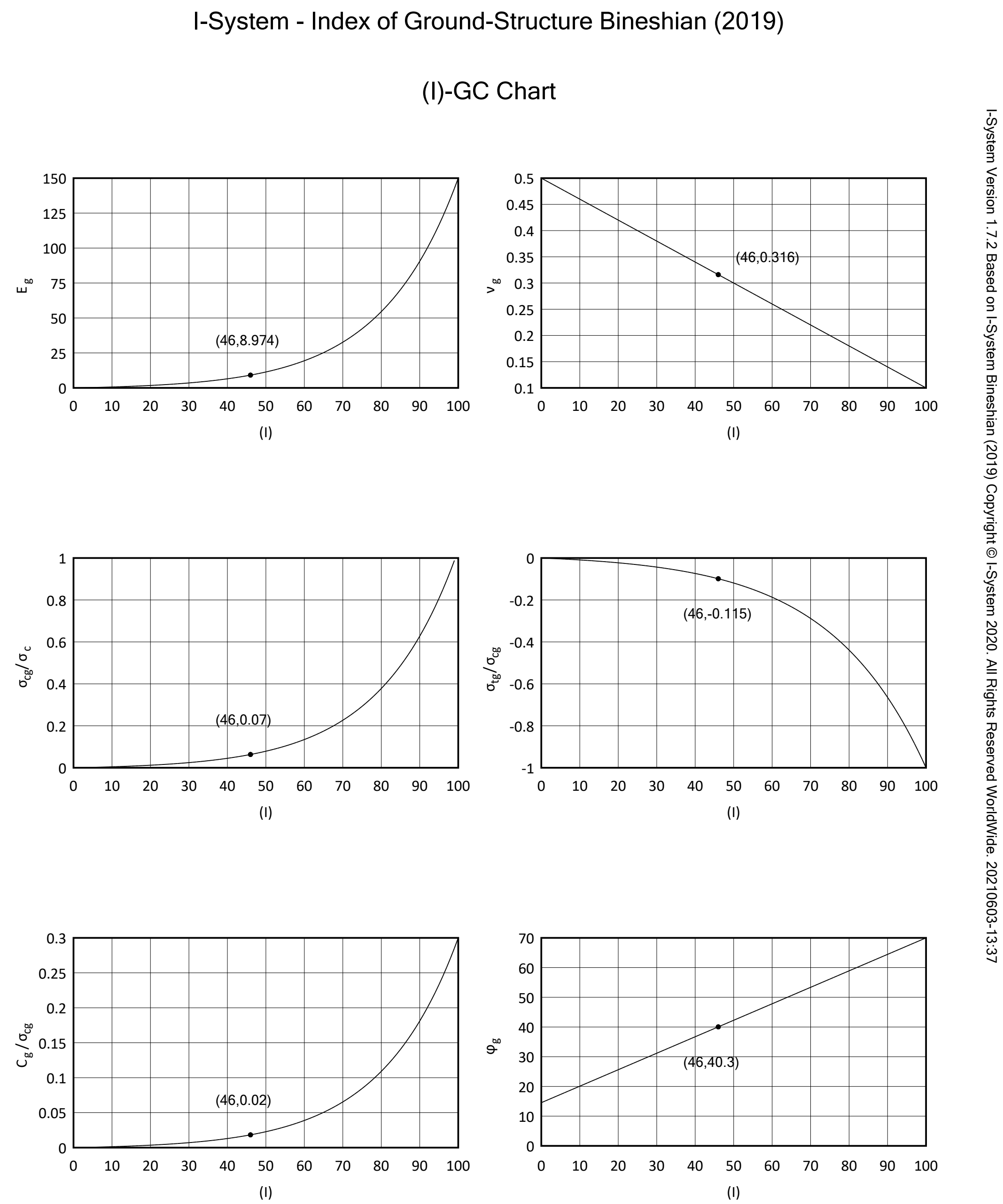

Figure 3d. I-System Software output – USBRL – T13 Tunnel

5. Semi-surface Unweathered Rock Works – Teesta HEP (Stage VI)

NHPC is the client of the project. Work is being executed by LTHPL. Subject work is the characterisation of ground for slope stabilisation design for barrage and power intake pile wall and the desilting basin. I-System is used for derivation of ground parameters as well as prediction for required measures for stabilisation of the subject structures.

Six parameters including E_g, σ_{cg}, σ_{cg}, C_g, and φ_g for the fresh rock mass for the semi-surface structure is estimated using (I)-GC.

Analysis input and output of I-System Software is presented in Figure 4.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi Bineshian (2019)

Calculation Sheet: Fresh	Location: Sikkim
Project: TeestaHEP	Type of Structure: Semi-Surface
Country: IN	Date: 2021/05/22

Ai - ARMATURE INDEX: 9.50

dn Discontinuity Number/s - per m	10 - 14
ds Discontinuity Set/s	3
di Discontinuity Inclination - °	31 - 60
da Discontinuity Aperture	Tight
dd Discontinuity Disintegration	Unweathered/Unaltered
df Discontinuity Friction	High Friction - Rough/Uneven
dp Discontinuity Persistency	< 0.90 x D

Ci - CONFIGURATION INDEX: 12.75

pc Problematical Configuration	Fractured - Moderately
sc Structural Configuration	Layered (> 100 cm)

Hi - HYDRO INDEX: 16.00

gc Ground Conductivity (GCD) || [Wetness]	(3 - 4.99) || [Moist]
gs Ground Softness - Mohs	≥ 7

Pi - PROPERTIES INDEX: 12.00

cc Cohesiveness Consistency	Indurated
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	N/A (e.g. Grainless)
pm Particle Morphology	N/A (e.g. Grainless)
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(3499 - 3000) || [1999 - 1500]

Si - STRENGTH INDEX: 12.60

cs UCS	74 - 50 MPa
se Scale Effect	B/H = 1.20 - 0.80

DFi - DYNAMIC FORCES IMPACT: 0.90

(PGASD) || [ERZ] || {MSK}	(0.26g - 0.35g) || [H] || {VII-VIII}

ETi - EXCAVATION TECHNIQUE IMPACT: 0.99

(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

Figure 4a. I-System Software output – Teesta HEP – Barrage

I-System - Index of Ground-Structure Bineshian (2019)

56%

(I)-Class

(I)-05

Recommended Measure/s

SS - Support System

Scng, SpotB32/SpotA32, HEAM/WeldM, DH54.L

ET - Excavation Technique/s

(PreS, DD6000-), (ProdBlast, PD3000-)

IT - Instrumentation Technique/s

3DMS@150m

PT - Prevention Technique/s

Protect Crest with FRS to Prevent Increment in Pore Water Pressure, Avoid: ProdBlast/UnCtldBlast, &
Bulk Removal of Toe

FT - Forecast Technique/s

ERT/VPH54.L

Design Remark/s

Check against plain/wedge/toppling failure & rock fall criteria, SFL not required

Figure 4b. I-System Software output – Teesta HEP – Barrage

I-System - Index of Ground-Structure Bineshian (2019)

(I)-GC; I-System's Ground Characterization

(I) = 56

Selected UCS range is 74 - 50 MPa.

Specified σ_c Value = 50 MPa

Modulus of Deformation

E_g = 15.445 GPa

Poisson's Ratio

ν_g = 0.276

Unconfined Compressive Strength

σ_{cg} = 5.756 MPa

Uniaxial Tensile Strength

σ_{tg} = -0.990 MPa

Cohesion

C_g = 189.298 KPa

Internal Friction Angle

φ_g = 45.800°

(I)-GC characterizes the ground based on (I); however, it is recommended to scrutinise it by deriving the mechanical properties of ground by standardised in-situ testing methods.

Figure 4c. I-System Software output – Teesta HEP – Barrage

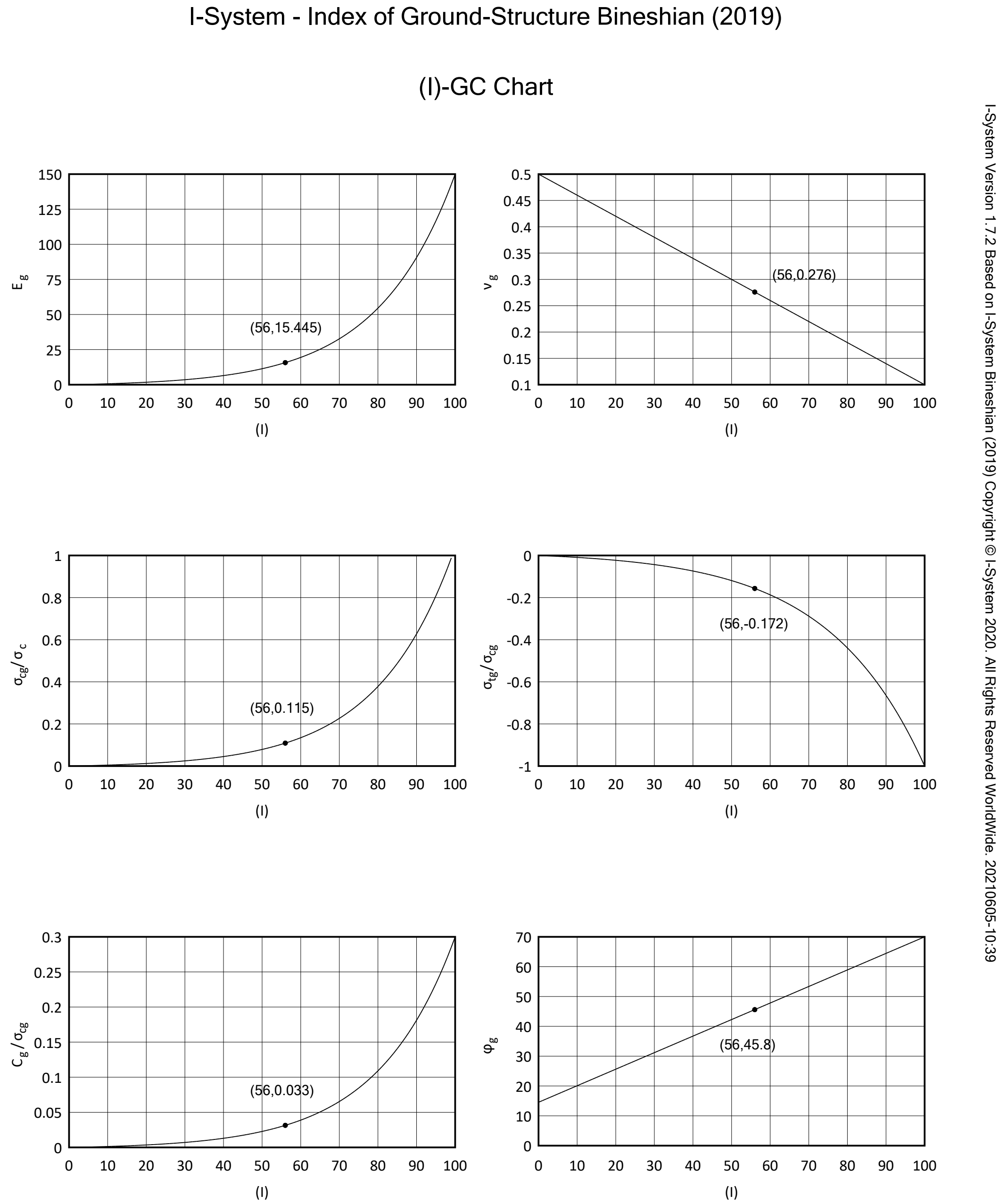

Figure 4d. I-System Software output – Teesta HEP – Barrage

6. Surface Highly Weathered Rock Work – Teesta HEP (Stage VI)

Subject work is the characterisation of ground for slope stabilisation design for barrage and power intake pile wall and the desilting basin in a highly weathered rock mass as a surface structure. I-System is used for derivation of ground parameters as well as prediction for required measures for stabilisation of the subject structures.

Similar to the Section 5, six parameters including E_g, σ_{cg}, σ_{cg}, C_g, and φ_g for the highly weathered rock mass for the surface structure is estimated using (I)-GC.

Figure 5 provides the input and output of the analysis conducted using I-System Software.

(I) = (Ai + Ci + Hi + Pi + Si) × DFi × ETi Bineshian (2019)

Calculation Sheet: Weathered	Location: Sikkim
Project: TeestaHEP	Type of Structure: Surface
Country: IN	Date: 2021/05/22

Ai - ARMATURE INDEX: 2.03

dn Discontinuity Number/s - per m	20 - 24
ds Discontinuity Set/s	≥ 4
di Discontinuity Inclination - °	31 - 60
da Discontinuity Aperture	Semi-Tight
dd Discontinuity Disintegration	Weathered/Altered
df Discontinuity Friction	Moderate Friction - Nonsmooth
dp Discontinuity Persistency	< 0.90 x D

Ci - CONFIGURATION INDEX: 9.00

pc Problematical Configuration	Fractured - Highly
sc Structural Configuration	Layered (100 - 10 cm)

Hi - HYDRO INDEX: 9.00

gc Ground Conductivity (GCD) || [Wetness]	(5 - 6.99) || [Leak]
gs Ground Softness - Mohs	6

Pi - PROPERTIES INDEX: 10.00

cc Cohesiveness Consistency	Indurated
dc Denseness Consistency	Never Indented by Thumbnail
ps Particle Size	N/A (e.g. Grainless)
pm Particle Morphology	N/A (e.g. Grainless)
bw Body Wave Velocity - m/sec (Vp) || [Vs]	(2499 - 2000) || [999 - 750]

Si - STRENGTH INDEX: 9.00

cs UCS	29 - 20 MPa
se Scale Effect	B/H = 1.20 - 0.80

DFi - DYNAMIC FORCES IMPACT: 0.90

(PGASD) || [ERZ] || {MSK}	(0.26g - 0.35g) || [H] || {VII-VIII}

ETi - EXCAVATION TECHNIQUE IMPACT: 0.99

(ET) || [PPV mm/sec]	(ME/NonExBreak) || [< 2]

Figure 5a. I-System Software output – Teesta HEP – Desilting basin

I-System - Index of Ground-Structure Bineshian (2019)

35%

(I)-Class

(I)-07

Recommended Measure/s

SS - Support System

Scng, SysA32.L.S, FRS250, PostG/I, DH54.L

ET - Excavation Technique/s

ME/NonExBreak

IT - Instrumentation Technique/s

3DMS@25m, IncM@400m

PT - Prevention Technique/s

Cover Slope Crest with WPM & FRS at a Width Equal to Height to Help Prevention of Tension Crack Generation, Avoid: ProdBlast/UnCtldBlast, Sharp/Tall Slope, Short Berm, Surcharge at Crest, & Toe Lightening

FT - Forecast Technique/s

ERT/SRT/VPH54.L

Design Remark/s

Check against plain/wedge/toppling failure & rock fall criteria

Figure 5b. I-System Software output – Teesta HEP – Desilting basin

I-System - Index of Ground-Structure Bineshian (2019)

(I)-GC; I-System's Ground Characterization

(I) = 35

Selected UCS range is 29 - 20 MPa.

Specified σ_c Value = 25 MPa

Modulus of Deformation

E_g = 4.755 GPa

Poisson's Ratio

ν_g = 0.360

Unconfined Compressive Strength

σ_{cg} = 1.007 MPa

Uniaxial Tensile Strength

σ_{tg} = -0.075 MPa

Cohesion

C_g = 11.590 KPa

Internal Friction Angle

φ_g = 34.250°

(I)-GC characterizes the ground based on (I); however, it is recommended to scrutinise it by deriving the mechanical properties of ground by standardised in-situ testing methods.

Figure 5c. I-System Software output – Teesta HEP – Desilting basin

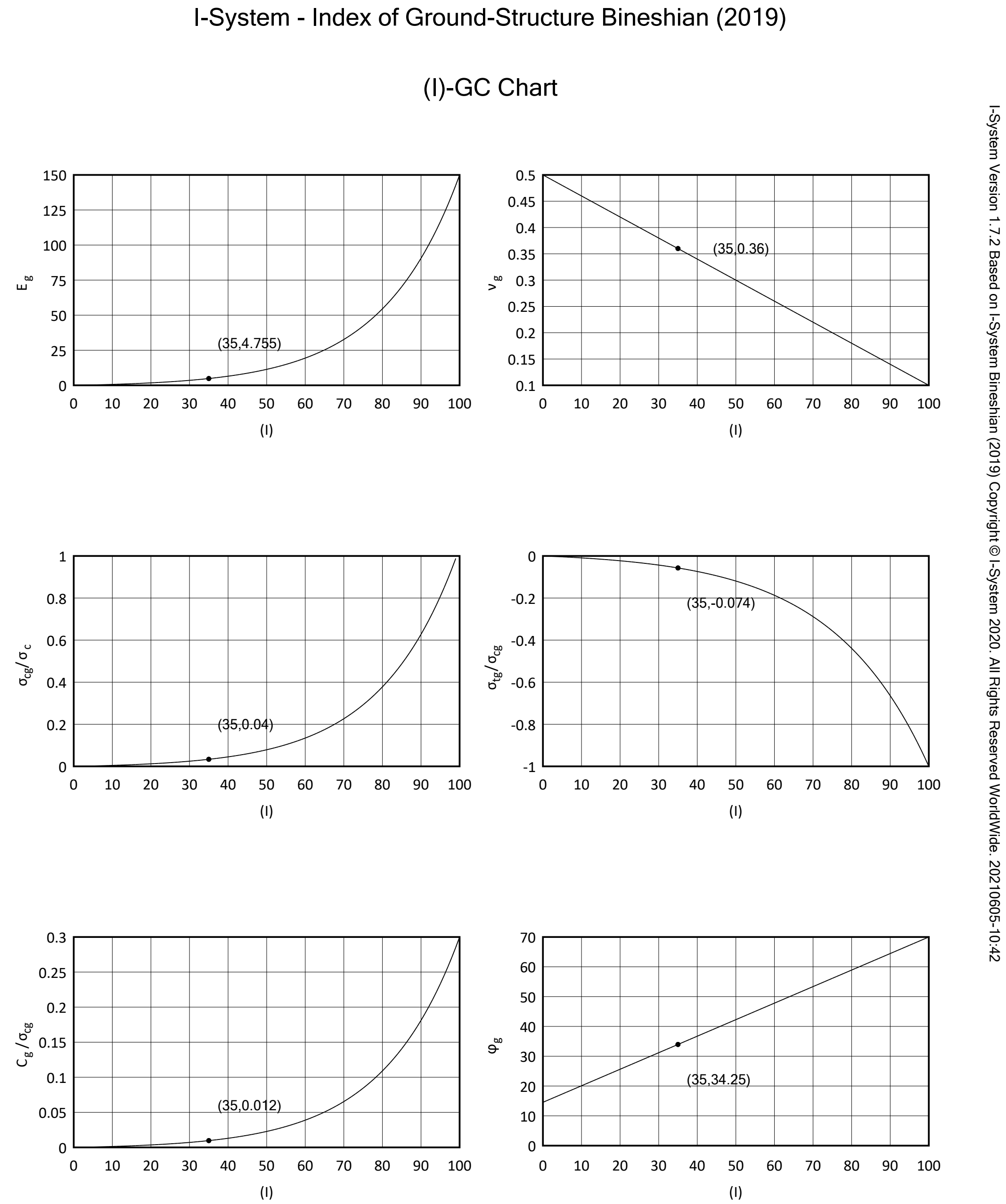

Figure 5d. I-System Software output – Teesta HEP – Desilting basin

7. Conclusions

I-System is used for classification as well as characterisation of ground in several projects before it is first ever published in 2019 and after that till now. Five case studies are presented here in this paper for the above-stated purpose in underground structures (T02, and T05, USBRL), semi-surface structure (Teesta barrage and distilling basin), and surface structures (T13 portal and Teesta barrage).

The results as output of I-System in both (I)-Class and (I)-GC is applied in both design as well as practice. Execution of some of these projects are already completed and some are being executed.

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Chronology of Development of I-System

A Brief History

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Abstract

This article provides chronology of development of I-System. The purpose to develop a new classification is explained and the aim to develop a comprehensive characterisation is presented. The general shortcomings of the existing classifications are briefly addressed and the reason to use I-System is clarified. Importance of comprehensiveness of an engineering classification system for both rocks and soils as well as varieties of tunnelling methods is described and necessity of consideration of most important features of ground as well as structures is briefed.

Keywords: (I)-Class, (I)-GC, I-System

1. Introduction

Empirical and observational design elements in a healthy design procedure is based on a suitable engineering classification and ground characterisation as the main part in design of structures in ground (Bineshian, 2019a, 2019b, 2020). It is necessary that the engineering classification to be comprehensive enough to be applicable for both rocks and soils and to be appropriate for application in modern tunnelling methods (e.g., NATM, NMT, SEM, SCL, etc.). Existing engineering classifications come with limitations in use, imprecision in regular application, and inaccuracy in estimation, which make engineers uncertain in determination and dimensioning of structures specially when they encounter ground complications (Bineshian, 2014, Bineshian, 2017a, Bineshian et al, 2019). RMR by Bieniawski (1973) and Q by Barton et al (1974) are widely used existing classifications; however, they lag in applications, comprehensiveness, accuracy, and precision. They are only applicable for rock medium; RMR is proposed for surface and underground works and Q for tunnels only. They take few parameters of rocks as input for the classification while very important parameters of ground are ignored, which have great impact on the ground quality. None of them considering excavation methods or properly taking the structures specifications (Bineshian, 2019b). I-System is developed to be used as a comprehensive classification and characterisation system for ground; i.e., both rocks and soils (Bineshian, 2019b). It is verified against varieties of ground conditions and scrutinised in several projects through 22 years research to address and resolve the aforesaid issues involved with existing classifications. I-System provides prediction of ground behaviour together with recommendations on required support system/s, excavation technique/s, instrumentation technique/s, prevention technique/s, and forecast technique/s followed by design remark/s as well as estimation for important mechanical properties of ground. Its output is optimised by analytical, numerical, and observational methods to compensate the demerits of existing classifications and strengthen its comprehensiveness.

2. Chronology

Table 1 represents a brief chronology of the development course of I-System. It is avoided to present all the details; however, main progress in research and the development is listed.

Table 1. Chronology and development procedure of I-System since 1999.

Year	Description
1999	- Initial form of (I) is developed as a function of uniaxial compressive strength, RQD, discontinuities' dip effect, rocks and soils' fabric, texture, and structure, and ground water condition.
2000	- Further development is attained by adding more important structural properties of rocks as armature index. - RQD is removed from the system due to its inefficiency in designating the rock quality. - Important easily derivable soils properties considering ground structural configurations is added to the system for better modelling of soils properties. - Dynamic forces' impact on the structure is added to the system for a better modelling of ground-structure reaction.
2001	- Further development is obtained by improvement of soil modelling in the system using morphology, cohesiveness consistency, and denseness consistency as some elements of properties index of the system.
2002	- Further development is achieved by adding the most important seismic properties of ground to the system to describe better the properties of rocks and soils.
2003	- Further development is applied to the system by establishing the structural as well as problematical configurations of ground including the tectonic features and structural geological aspects of ground; this part is called configuration index of the system.
2004	- I-System was further developed by considering the negative effects of water on ground including the effects in softening of the ground materials as well as the pressure effect both as main elements of hydro index of the system. - Excavation method's impact is added to the system by considering the method of excavation and peak particle velocity.
2005	- Seismic as well as dynamic effects including peak ground acceleration's effect on the structure is further developed as impact factor for proper modelling of the structure in relation to the ground.

2006	-	Scale effect and consideration of shape and depth of placement of the structure is considered in the system as elements of the strength index.
2007	-	Indices and impact factors are further assessed and the derivation of parameters are tested in practice.
2008	-	Hydraulic conductivity of ground is included in the system as part of hydro index and the system is further developed to approach its final form.
2009	-	(I)-Class's development is initiated as I-System's classification system and its initial form for underground, semi-surface, and surface structures is prepared.
2010	-	Special ground conditions including burst prone, squeezing, swelling, heaving, gravity driven, and time dependent grounds/behaviour are taken into consideration to be included in the (I)-Class.
	-	I-System is further verified in practice.
2011	-	Development of Stress Release Holes (SRH) for squeezing, swelling, and heaving (SSH) ground is initiated.
	-	I-System is further verified in various ground conditions; consequently, corrections/amendments in scoring system is applied.
2012	-	Further verification of I-System and its (I)-Class is conducted in practice.
	-	Scoring system is further assessed for the parameters of the indices and impact factors.
	-	Classification for SSH condition is initiated.
2013	-	Development of Ground Conductivity Designation (GCD) is initiated as a criterion for assessment of ground conductivity as well as solidification quality.
	-	I-System is further verified in practice.
	-	(I)-Class's recommendations were assessed in details for different structures and its recommendations were further improved.
	-	Further development on SRH is obtained.
2014	-	GCD is tested in practice and it is further assessed.
	-	I-System's scoring system is further modified.
	-	(I)-Class's recommendations are further improved.
	-	SRH research is reached to its final form to be applied in practice.
2015	-	SRH is applied in practice and the results are assessed.
	-	Further work is conducted on I-System's scoring system to form its final format.

	- (I)-Class's recommendations are verified for challenging tunnelling condition. - GCD is further tested in practice.
2016	- Development of (I)-GC as I-System's ground characterisation for estimation of mechanical properties of ground is initiated. - I-System and (I)-Class are reached to their final form while they are further verified in practice. - SRH is successfully applied in practice.
2017	- GCD's results cited in publications (Bineshian, 2017a, 2017b, Bineshian, et al, 2019). - Further researches on SRH are performed. - I-System is further verified in practice. - (I)-GC's accuracy is further improved by derivation of best fit on empirical data.
2018	- I-System is further verified; it is formally used in design. - GCD and SRH have obtained their final forms. - (I)-GC is further developed while it has obtained acceptable accuracy in estimation of mechanical properties of ground.
2019	- I-System is first ever published (Bineshian, 2019a, 2019b) while it is further applied in design and its performance was verified systematically. - GCD and SRH are further applied in practice and their suitability were verified. - Research on ViD (vibration-induced damage) is initiated as a function of (I). - Development of empirical equations/methods for estimation of pull length in underground works as well as rock bolting system calculation based on I-System is initiated. - (I)-GC is included in I-System as a comprehensive characterisation system.
2020	- ViD, pull length advisor, and systematic bolting calculator are developed based on (I). - GCD is published (Bineshian, 2020a). - SRH System is published (Bineshian, 2020b). - I-System is further verified. - I-System Software is developed (Bineshian, 2020c).
2021	- ViD is published (Bineshian, 2021). - 2021 edition of I-System is in press containing further clarifications as well as added features; application of I-System is eased. - I-System Software is further developed to include all added features of I-System.

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Long Range Underground Prediction of Ground Behaviour/Hazards at Tunnel T13 in USBRL Project using TSP

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Abstract

Estimates of uncertainties and its associated risk during the construction process are essential information for decision making in any stage of a tunnel project. Dealing with adverse ground condition at any depth can be problematic, which will lead to a significant delay and cost upsurge if it is not adequately predicted. The amount of information available for the ground can be increased by specialised geophysical technique, which provide indirect assessment of engineering properties. The aim of such type of geophysical investigation is to model the expected behaviour of the ground and thus to predict the scenarios indicating potential variations in the quality of the rock mass during underground excavation. Tunnel Seismic Prediction (TSP) as an advanced geophysical method for estimating ground condition ahead and around of unexcavated areas in tunnelling quantifies mechanical ground parameters based on body waves velocities (compression and shear wave velocities). Seismic velocities are sensitive to rock mass quality, porosity, stress state, and water condition. The method is widely accepted because of its long prediction range and high resolution. TSP 303 is used in tunnel T13 of Udhampur Srinagar Baramulla Rail Link (USBRL) Project in J & K, India and achieved a remarkably high prediction accuracy of 90 per cent. The prediction of ground condition at T13 helped to prevent potential failure/collapses to occur. In this article the use of seismic properties of ground to assess the geomechanical behaviour ahead of the face of the tunnel is illustrated.

Keywords: GCD, I-System, SRH, tunnelling, TSP

1. Introduction

Geophysical investigations in underground engineering imply a series of geophysical methods. The tunnelling industry has already identified the potential of these non-destructive methods that valuably contribute to the assessment of the ground condition and to the provision of an interpretative prediction guideline for advancement. Due to long range prediction and high resolution, seismic methods are classified as optimized methods in which they are usually preferred for the assessment of the ground condition/s.

Many tunnels are located in areas with relatively weak access along the alignment and excavated/bored under extremely high overburden. These two factors often result in limited geological information. It would be reasonable to state that the deeper the tunnel, the greater the level of uncertainties due to less accurate geotechnical data available.

The normal approach to assess the geomechanical condition is to obtain the geological section along the tunnel by observation, borehole drilling, and surface geophysical survey. However, factors like overburden, thickness, and topography may limit the potential of these methods to obtain the precise and sufficient information. In excavation through inaccessible mountain even an extensive geotechnical baseline report can miss the critical ground surprises. This can create a big challenge for both drill and blast and TBM drive in terms of their performance and safety.

The safe operation can be achieved by implementing the right method. The overall goal should be minimising the risk in such a way that it always is within acceptability (Dickmann, 2013, Dickman and Krueger, 2014, Dickmann et al, 2018). The only way to achieve the acceptability of risk is to control them. Knowing in advance where the significant geological boundaries intersect the tunnel axis can help to prevent hazards such as large failures, collapses, and extreme ground conditions.

2. Overview of Tunnel Seismic Prediction

The TSP is based on the evaluation of elastic body waves, which are being excited by detonation charges providing the best signal to noise ratio and the least restrictive conditions for recording and processing (Figure 1).

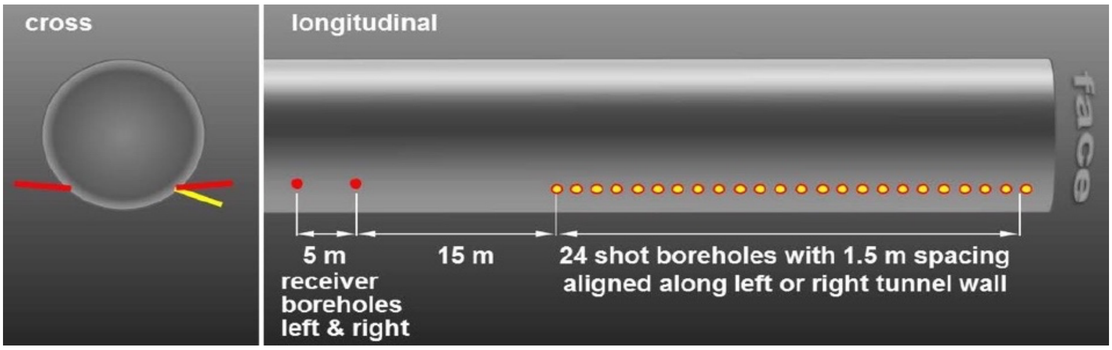

Figure 1. Measurement layout of the 3D Tunnel Seismic Prediction technique (TSP 303) consisting of usually 4 receivers (RCV) and 24 shot points

The body waves travel as compression or shear waves through the ground and are being reflected at interfaces with different mechanical properties like density or elasticity. Thus, by separation of the different wave types using three-component-sensors it is possible to derive information about the mechanical properties of the ground such as dynamic modulus of elasticity. Even in rather complex geomechanical hard rock condition, prediction ranges of 100 - 150 m can be achieved.

Acoustic signals are produced by a series of 24 shots, containing 25 to 100 grams of explosives. Four sensor probes, consisting of highly sensitive tri-axial receivers, are contained in protection tubes whose tips are firmly cemented into boreholes of 45-50 mm in both side-walls (Figure 1). The 3-component receivers pick up the seismic signals, which have been reflected from any kind of discontinuity in the ground ahead.

A highly sophisticated processing and evaluation software has been devised for ease of operation. The capability of the system to record the full wave field of compressional and shear waves in conjunction with the intelligent analysis software enables a determination of ground mechanical properties such as Poisson Ratio and Young's Modulus within the prediction area.

3. Specific Aspect of Tunnel Seismic Operation

The necessary operations to perform a tunnel seismic measurement in a typical TSP setup can be integrated into the construction operations without any interference with the excavation work. Boreholes for receivers can be prepared using ordinary drilling systems. Explosive charging can be conducted as simple as tamping of a single cartridge in a short hole.

Installation of seismic receivers as well as charging and shooting of holes may take place during maintenance intervals or short excavation breaks of about one hour. This operation time can be further reduced by splitting the campaign into two parts that can be carried out on consecutive days.

For advancing of a long and deep tunnel, the decision had been usually made for the use of a TBM. There is even a tendency to specify a shielded machine when in fact an open machine may do the job. Here, the use of precast segments will constitute a crucial point because it shall limit seismic surveys since the ground is not accessible at all. In order to avoid large-scale drilling measures through the precast segments, it would be very helpful to use the grouting and lifting inserts of the segments. For example, the hexagonal or honeycomb segmental lining provides a quick and easy layout of the seismic bore line.

Regular grouting inserts every 1.5 meters fit perfectly to the regular spacing of the seismic layout (Figure 2). The stability, safety, and the serviceability of segmental elements are guaranteed using explosives for TSP measurements.

In case of full backfilling of the segments the blasts could activate settlements with a maximum of 3 mm in worse ground strengths like weathered mudstone. The settlements become less with increasing ground strength.

Damage-free blasts can be performed if the blowouts are canalized by installed tubes, while the blow out plane behind the segments is concurrently eliminated. It can be stated that TSP is applicable for TBM drive with segmental lining where any damage to lining elements due to the required explosive charges can be excluded.

Figure 2. Installation of TSP sensor into 2 m deep inside the side wall - NATM (left) and TBM (right)

4. Case Study

Himalayan mountain range is arc-shaped, convex southwards with syntaxial bends at the western and eastern ends (Figure 3). The syntaxial western bend is parallel to a continental scale deep fault known as the Chaman Fault. The Himalayan Mountain range is subdivided into four principal tectonic zones, from south to north these are: Sub-Himalaya, Lesser Himalaya, Higher Himalayan Crystalline, and Tethyan Himalaya. Himalayas are known to be very seismically active and the number of earthquakes has been recorded in historical times. Tunnel T13 project is located in the state of J&K and alignment passes through highly undulating and steep hill slopes of the younger Himalayas.

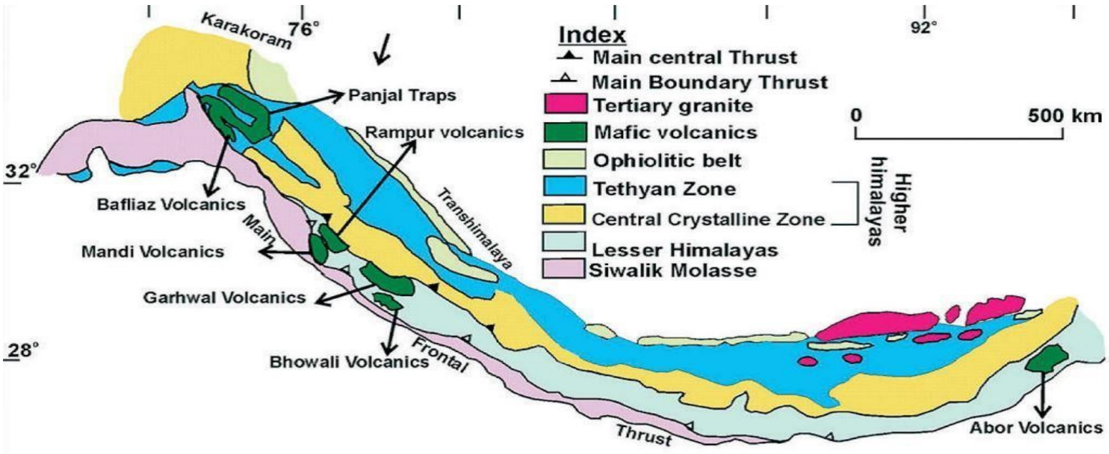

Figure 3. Outline geological map of the Himalayan Mountain Belt (Ahmed, 1988)

Table 1. Chronological order of Geological formation of the USBRL project area.

Group/Formation	Age	Lithology
Quaternary Deposits	Sub-recent to recent (Pleistocene to Holocene)	Terrace deposits, scree/ debris, slope wash, river borne material and alluvial soil.
Murree formation	Eocene to Miocene	Purple to reddish coloured sandstone, siltstone and claystone.
Sabathu Formation	Paleocene to Eocene	Variegated shale of Khaki, olive green and pale/yellow colour interlayer with calcareous sandstone, shale and nummulitic limestone
Jangalgali Formation	Crestaceous-Eocene	Chert/Quartz breccia, ferruginous sandstone/shale/ and pisolitic/non pisolitic bauxite
Sirban Group – Khairikot Formation	Meso-Proterozoic	Quartzite, dark grey slate and variegated shale with stromatolitic limestone/dolomite bands.
Sirban Group – Trikuta Formation		Dark grey to light grey dolomite, stromatolitic dolomite, slate, quartzite and subordinate limestone

The area falls in Seismic zone V of the standard seismic zoning map of India. Present alignment passes through terrain of rugged morphology occupied by round or sub round crested ridges and hills. The project alignment for Tunnel T13 passes through the Lower Murree formation of Upper Eocene age. It comprises of purple, brown to greyish alternate bands of medium to fine grained sandstone, siltstone, and claystone. Rock mass is highly fractured, sheared, and jointed in nature with the presence of numerous bands of pseudo-conglomerate. These rocks are much prone to weathering and erosion. On account of high tectonic activities in Himalayas, the rocks along the alignment are folded, over-thrusted and faulted at many places, resulting into highly jointed and crushed rocks. In this regard, Figure 3 is representing the anticipated L-Profile of the project T13 including three main geological zones and project line.

Government of India planned a 326 km railway line to provide an alternative and a reliable transportation system to state of Jammu & Kashmir (J&K) with the Indian Railway network from Jammu to Baramulla. The project has been declared as a Project of National Importance. Jammu-Udhampur-Katra-Quazigund-Baramulla Railway line is the largest project in the construction of a mountain railway since independence of India (Figure 4). Udhampur-Srinagar-Baramulla Rail Link (USBRL) is a mega project for construction of main part of above-mentioned railway line. It passes through young Himalayas with tectonised zones including major thrust faults.

Figure 4. USBRL Project Layout

The USBRL is in various stages of progress in the balance length from Katra to Banihal. The client for the project is Northern Railway (NR) as one of the 16 and of course largest route kilometres railway zone of Indian Railway (IR).

Tunnel T13 is located in the state of J&K and alignment passes through highly undulating and steep hills of younger Himalaya and through the Murree formation of upper Eocene age. It includes twin tunnels comprising of Main Tunnel (MT) and Escape Tunnel (ET) together with 24 Cross Passages (CPs) as part of USBRL Project.

The T13 has been assigned to Konkan Railway Corporation Limited (KRCL), which is a Union Government Company headquartered in Mumbai. Conduction of TSP is awarded to AMBERG Engineering headquartered in Switzerland with an Indian branch in Gurgaon.

This paper focuses on the TSP measurement on tunnel T13P1 ET between CH62898 to CH63076 as shown in Figure 5.

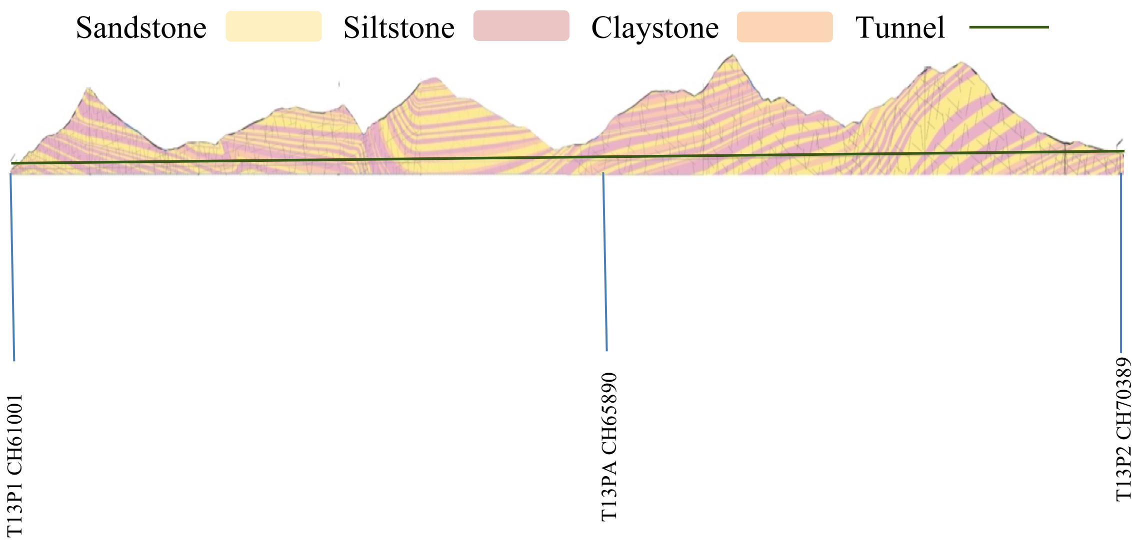

Figure 5. Location of TSP campaign & Geological L-Profile of Tunnel T13

Table 2 presents an overview of Tunnel T13's features.

Table 2. Salient features of T13 Tunnel

Type	Twin (MT & ET)
Total Length (MT)	9.37 km
NATM Portion	9.37 km
Total Excavated Length	3.67 km
Water Inflow	Dripping in most of the length
Lithology	Sandstone, Siltstone & Claystone
Geo-structure	Techtonised - Moderately to Highly Jointed

Geotechnical/geomechanical observations of measurements is summarised as follows:

- Surrounding ground's main material: Siltstone
- Overburden ≈ 150 m
- No of JS: 3+
- RQD < 55
- UCS of Intact Rock ≈ 70 MPa
- UCS of Rock Mass < 7 MPa
- (I)-05 – (I)-06 (as per I-System; Bineshian (2019a, 2019b, 2020a))
- Water condition: Dripping (GCD = 6 – 15; Bineshian (2020b))
- Infilling of the discontinuities: Gouge comprising silt and clay
- Orientation of discontinuities: Unfavourable

- Vp: 4000 – 5600 m/sec (Primary Wave Velocity); Figure 5
- Vp: 2200 – 3100 m/sec (Primary Wave Velocity)
- ERZ: VH with MSK IX-X (Earthquake Risk Zone)
- SS: FRS (Fibre Reinforced Shotcrete), SysRB (Systematic Rock Bolting), and SRH (Stress Release Holes; Bineshian (2020c))
- Mechanical behaviour of surrounding ground: Minor – mild squeezing
- Excavation Technique (ET): Full face

Figure 6 shows the 2D rock property chart and plan view of dynamic young's modulus along seismic axis. Red colour at chart of Dynamic Young's Modulus generally indicates reduced rock stiffness whereas blue colour indicates enhanced rock stiffness.

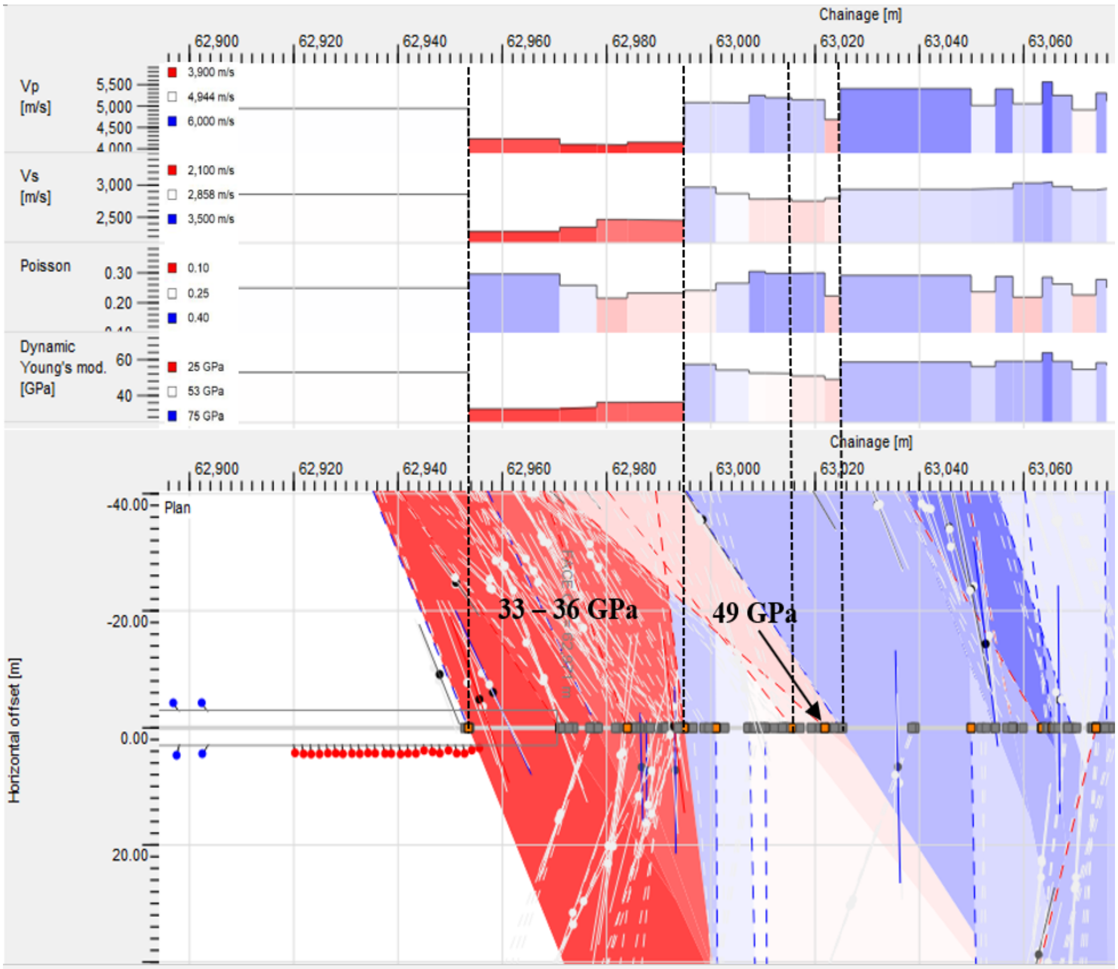

Figure 6. Ground property chart and plan view of dynamic young's mod along seismic axis

The estimated reference value of dynamic Young's Modulus (E_{dyn}) is 53 GPa and it varies between 33 GPa to 64 GPa along the tunnel axis. Directly ahead of tunnel face, E_{dyn} decrease abruptly below the reference value. In addition to this, the distribution of P –wave velocity is shown on Figure 7.

In some section ahead of face, P-wave velocity is below the reference value i.e., 4,433 m/s, which allows inferring that at some of these sections decrease in rock stiffness might be expected.

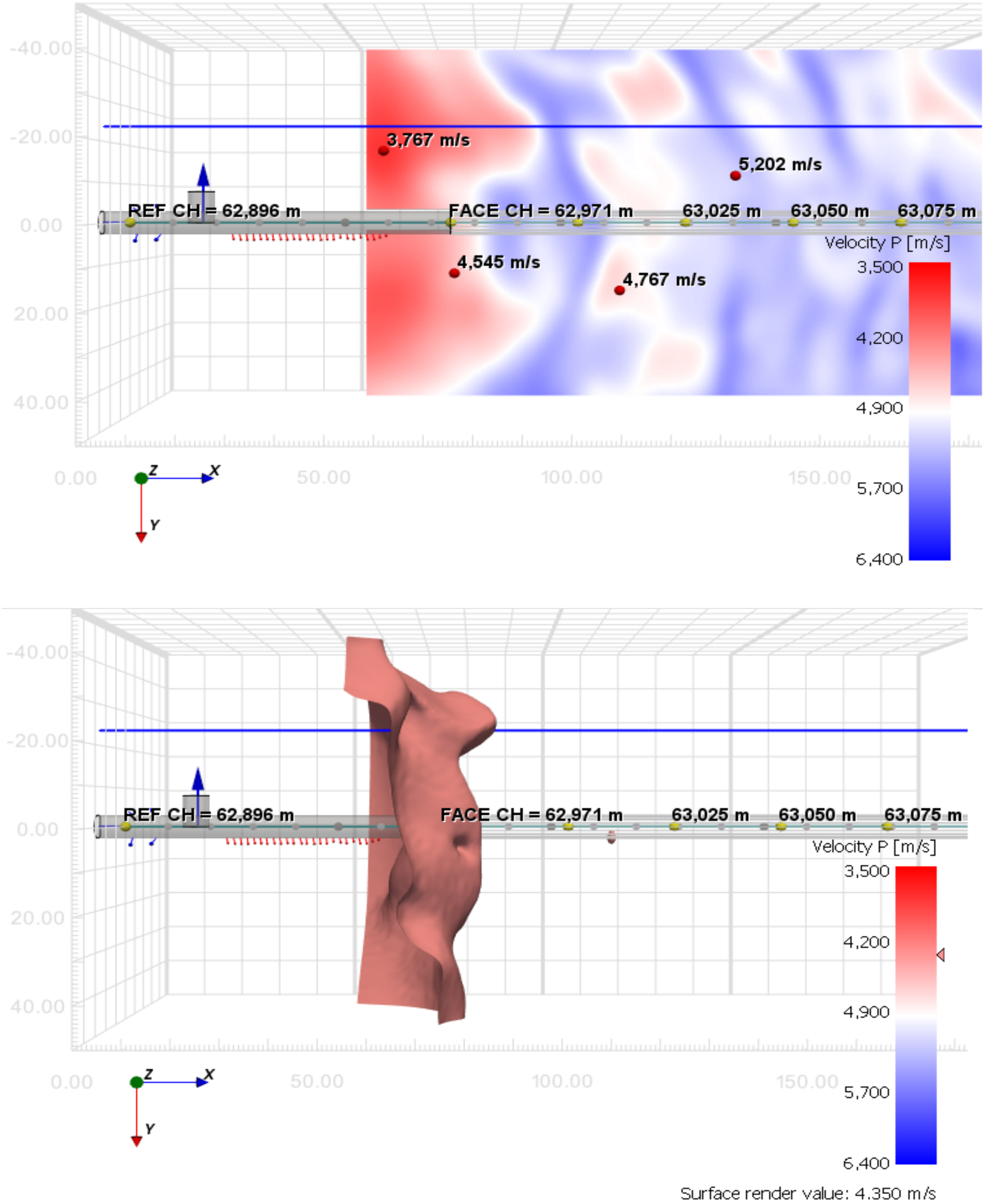

Figure 7. Top view of 3D P wave velocity model along MT as well as ET axis (top) and surface rendering of 3D-P wave velocity lower than 4,350 m/s (bottom)

In addition to predict the behaviour of rock mass, it is also possible to explore the possibility of water bearing zones on the basis of variation in poison ratio which is being calculated from primary & shear wave velocity. Figure 8 shows the distribution of rendered Poisson's ratio of $\sigma \geq 0.29$ within the prediction range around & ahead of the tunnel axis.

Depending on the seismic response at a given site, Poisson's ratio greater than equal to 0.29 are assumed as good indicators of these possible water bearing zone.

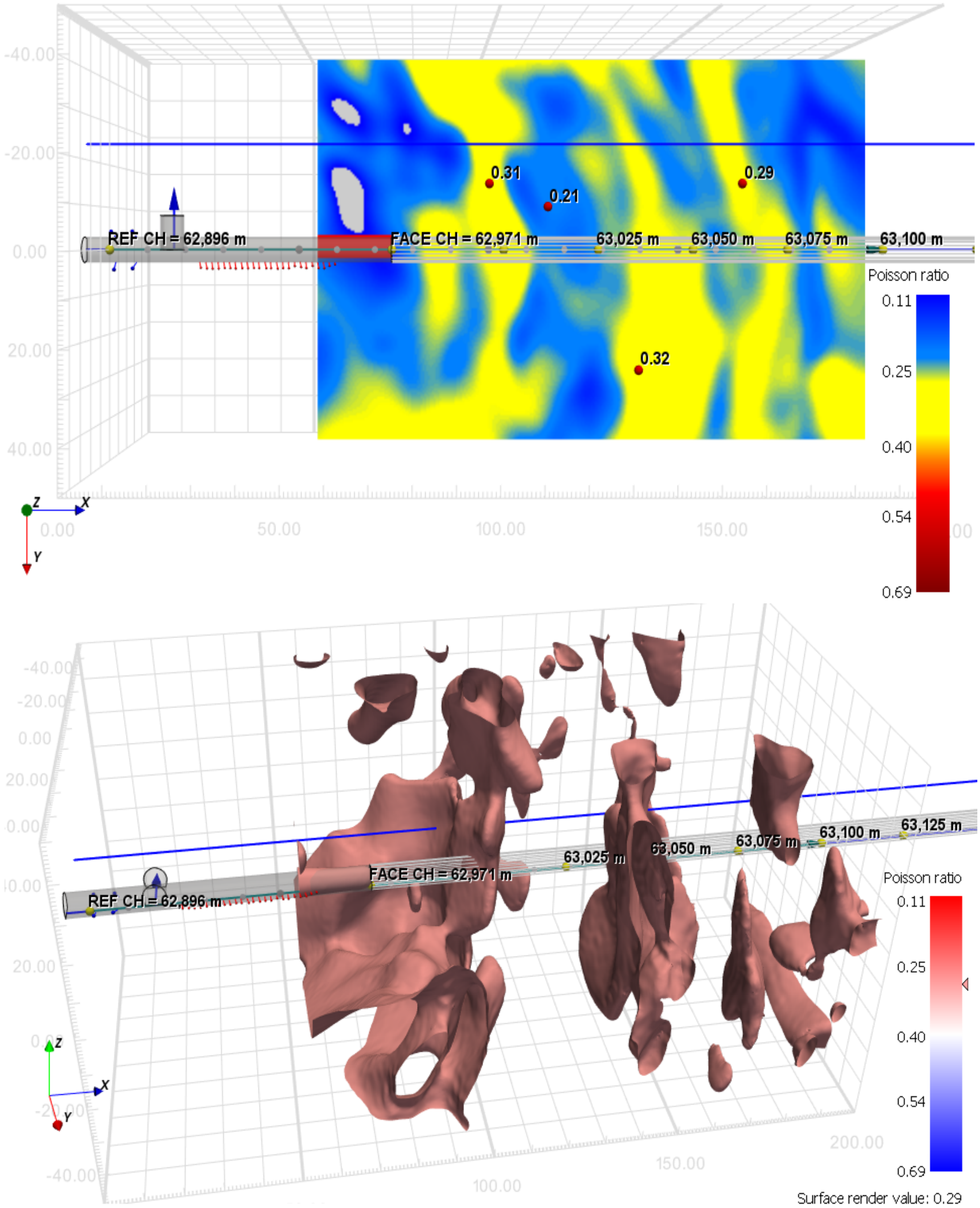

Figure 8. Distribution of poison ratio along MT as well as ET axis (top) and 3D view with surface rendering value of 0.29 (bottom)

TSP has been conducted at CH 62954 and a length of 122 m ahead was predicted. TSP has predicted the ground using I-System (Bineshian, 2019a, 2019b, 2020a) as weak to fair rock mass (including (I)-05 to (I)-08 as per I-System's classification) ahead of above-said chainage.

TSP results were compared with actual gathered data from site during excavation and a 90 per cent matching is obtained. Appendix 1 presents TSP results and Appendix 2 provides a comparison conducted by Client and Contractor's Engineers/Geologists. As can be seen mismatching is only 10 per cent of the length, which is not also deviated from the quality predicted; however, it has given more conservative info compared to the actual data obtained.

On the basis of comparison of TSP prediction with actual ground condition encountered, calibration of seismic parameters for a given site can be initiated. A valid calibration will certainly increase the accuracy while doing data analysis in further measurements. So, it is always necessary to conduct the TSP measurements on a regular interval which enable Client/contractor to tackle any further ground surprises.

TSP procedure consumed only 0.12 per cent of drive in such challenging condition in tunnel. Hindrance caused by TSP procedure is almost nothing (≈ 1 hr 35 mins for 122 m length of tunnel) while the prediction is tuned to have 90 per cent matching with actual condition encountered.

Tunnelling in the initial stretch was an exceptionally challenging due to weak surrounding ground and water bearing zone; however, due to having high accuracy prediction no failure or collapse happened and therefore based on the prediction provided, prevention techniques are applied to prevent occurrence of any type of gravity or lateral failure or even caving. It is strongly recommended to tune the TSP results by conduction of GCD (Bineshian, 2020b), EC (Exploratory Coring), or BH (Blind Hole Probing) for a safe drive in tunnel.

5. Conclusions

As per the stated comparison, TSP prognosis has close correlation with actual ground condition encountered including ground class and water condition. The prediction of the ground condition at T13 helped the client and contractor to prevent potential failure and collapses to occur in the tunnel with an accuracy of at least 90 percent.

Geophysical methods are an essential part of modern tunnelling, which enables continuous risk assessment and management during construction. They are meaningful and necessary tools in modern tunnelling and it is well noted that the tunnelling community continuously overcomes its scepticism and doubts about the potential of these methods. When exactly realising the optimal use of them, tunnelling will become more predictable in both costs and risks.

TSP can be the right way to turn the geomechanical risk/s and hazard/s into manageable condition/s. This advance technology can give a project the required support in overcoming the risk associated with geomechanical uncertainties.

The TSP operation does not make any disturbance to tunnelling work if operated systematically. As it is presented in this paper, it only took 0.12 per cent of progress time for the length of prediction. To increase the accuracy, such type of geophysical techniques should be used in a regular manner.

Globally, TSP is a well-established geophysical technique for ground prediction in NATM or conventional methods of tunnelling comprising of mechanized or drill and blast excavation technique/s including full face boring systems or partial sequential digging/excavation techniques.

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Appendix 1. TSP Results for T13P1 ET

Table 3. Detailed results of TSP for T13P1 ET including measured physical and predicted mechanical properties

CH (m)	V_P (m/s)	V_S (m/s)	ν	E (GPa)	TSP Interpretation
62898 – 62954	4,944	2,858	0.25	53	**Reference rock stiffness** (Siltstone, (I)-06 and water bearing)
62954 – 62971	4,234	2,279	0.30	33	Decrease in rock stiffness (I)-08 – (I)-09 and possible water bearing zone
62971 – 62978	4,105	2,343	0.26	33	Decrease in rock stiffness (I)-08 - (I)-09 and possible water bearing zone
62978 – 62984	4,092	2,465	0.22	36	Decrease in rock stiffness (I)-08 - (I)-09 and possible water bearing zone
62984– 62995	4,152	2,452	0.23	36	Decrease in rock stiffness (I)-08 - (I)-09 and possible water bearing zone
62995 – 63001	5,081	2,968	0.24	57	Almost similar to reference, (I)-06
63001 – 63007	5,075	2,868	0.27	54	Almost similar to reference, (I)-06
63007 – 63010	5,249	2,783	0.30	52	Similar stiffness to reference, (I)-06 – (I)-07 and possible water bearing zone
63010 – 63015	5,195	2,786	0.30	52	Similar stiffness to reference, (I)-06 – (I)-07 and possible water bearing zone
63015 – 63022	5,148	2,752	0.30	51	Similar stiffness to reference, (I)-06 – (I)-07 and possible water bearing zone
63022 – 63025	4,682	2,797	0.22	49	Decrease in rock stiffness, (I)-06 - (I)-07
63025 – 63050	5,406	2,934	0.29	59	Increase in rock stiffness, (I)-05 - (I)-06
63050 – 63055	5,014	2,945	0.24	56	Increase in rock stiffness, (I)-05 - (I)-06
63055 – 63058	5,401	2,947	0.29	59	Further increase in rock stiffness, (I)-05 - (I)-06
63058 – 63064	5,053	3,032	0.22	59	Increase in rock stiffness, (I)-05 - (I)-06
63064 – 63066	5,564	3,050	0.29	64	Increase in rock stiffness, (I)-05 - (I)-06 and possible water bearing zone
63066 – 63069	5,249	2,978	0.26	59	Increase in rock stiffness, (I)-05 - (I)-06
63069 – 63074	4,912	2,923	0.23	55	Slight decrease in rock stiffness, (I)-05 - (I)-06
63074 – 63076	5,303	2,943	0.28	58	Increase in rock stiffness, (I)-05 - (I)-06 **[End of Prediction]**

(I)-Class I-System's Classification (Bineshian, 2019a, 2019b, 2020a)
CH Tunnel Chainage
V_P Body Wave – Primary Wave Velocity
V_S Body Wave – Shear Wave Velocity
ν Poisson's Ratio
E Dynamic Young's Modulus

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Appendix 2. Comparison of TSP vs Actual Condition for T13P1 ET

Table 4. Geomechanical comparison for TSP's resulted prediction and observed condition during excavation at T5 Tunnel

CH (m)	TSP Interpretation	Actual Condition/s		Matching Comparison
		(I)-Class	Water Condition	
62898 – 62954	(I)-06 + Water **(Reference)**	(I)-08	Dripping	100%
62954 – 62971	(I)-08 – (I)-09 + Water	(I)-08	Dripping	100%
62971 – 62978	(I)-08 - (I)-09 + Water	(I)-08	Showering	100%
62978 – 62984	(I)-08 - (I)-09 + Water	(I)-08	Showering	100%
62984– 62995	(I)-08 - (I)-09 + Water	(I)-08	Showering	100%
62995 – 63001	(I)-06 + Water	(I)-07	Dripping	(I)-06 predicted, (I)-07 observed
63001 – 63007	(I)-06 + Water	(I)-07	Dripping	(I)-06 predicted, (I)-07 observed
63007 – 63010	(I)-06 – (I)-07 + Water	(I)-07	Dripping	100%
63010 – 63015	(I)-06 – (I)-07 + Water	(I)-06 - (I) - 07	Dripping	100%
63015 – 63022	(I)-06 – (I)-07 + Water	(I)-06 - (I) - 07	Dripping	100%
63022 – 63025	(I)-06 - (I)-07	(I)-06	Damp	100%
63025 – 63050	(I)-05 - (I)-06	(I)-06	Damp	100%
63050 – 63055	(I)-05 - (I)-06	(I)-06	Dripping	100%
63055 – 63058	(I)-05 - (I)-06	(I)-06	Dripping	100%
63058 – 63064	(I)-05 - (I)-06	(I)-06	Dripping	100%
63064 – 63066	(I)-05 - (I)-06 and possible water bearing zone	(I)-06	Dripping	100%
63066 – 63069	(I)-05 - (I)-06	(I)-06	Dripping	100%
63069 – 63074	(I)-05 - (I)-06	(I)-06	Dripping	100%
63074 – 63076	(I)-05 - (I)-06 [**End of Prediction**]	(I)-06	Flowing	100%

(I)-Class I-System's Classification (Bineshian, 2019a, 2019b, 2020a)

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Earthquake scenario selection of Tindharia landslide in India

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Abstract

Earthquake-induced landslides can affect people and structures as a result of significant ground shaking or regardless of the intensity. A suitable design ground motion helps to mitigate the impact of such landslides. The most commonly used design ground motion in slope stability analysis is based on probabilistic seismic hazard maps. Uncertainties in the selection of expected ground motion levels have been ignored. The present study is conducted based on improvised fully probabilistic approach, which provides the total probability of slope failure in a particular period under seismic loading by addressing all possible scenarios. This approach is applied to the seismically active Tindharia slope located in Darjeeling, India. The total probability of seismic slope failure obtained in the next 50 years is 30% and the most probable peak-ground acceleration that triggers a landslide is 0.12g. Design peak-ground acceleration predicted from the next 475-year probabilistic seismic hazard map is 1.02g. In the present study, the significant difference in the design peak-ground acceleration from probabilistic seismic-hazard analysis and fully probabilistic approach is observed. The study suggests that the seismic landslide hazard may be overestimated or underestimated when used in the design of ground intensity obtained by conducting PSHA.

1. Introduction:

Earthquake-induced landslides pose major threats worldwide, especially in mountainous zones (Liao and Lee, 2000). India has experienced several destructive earthquakes ($M_w \geq 8.0$) that have caused deadly landslides, which have had an impact on the human environment in many ways. In this country, 70% of highly vulnerable landslides have been observed in the Himalayan region, Northeast India, Eastern Ghats, and Western Ghats.

Several methods and evaluation studies are proposed to determine the landslide hazard and conduct a susceptibility assessment (Guzzetti et al., 1999). The methods are either quantitative or qualitative approaches based on knowledge, experience, numerical expressions, methods, and computer-based models. Seismically induced landslide analysis considers not only the well-measured material properties but also proper ground-motion selection.

Many researchers have developed seismically induced hazard maps in terms of susceptibility or probability on a regional or global scale. Most of the hazard maps are developed by estimating slope parameters using a global information system that provides the rough hazard level of the specific site. The statistical and deterministic approaches focus on landslide susceptibility that helps determine where landslides are likely to occur through the use of physically based models regardless of triggering conditions (Van Westen et al. 2008; Lee et al., 2008). In current engineering practice, the quantified risk levels of the regional or global scale have been identified using hazard maps produced by probabilistic approach (Raghukanth and Iyenger, 2007). The triggering peak ground acceleration (PGA) in the slope stability analysis has been computed from probabilistic hazard maps. Most commonly, the PGA that initiates failure of the slope is measured from 475 years of 10% probability seismic hazard maps (Wang et al., 2017). The hazard maps developed based on an earthquake catalogue (EC) are not restricted to the use of regional ground-motion prediction equation (GMPE) only. The pseudo probabilistic, statistical, and deterministic approach provides a conservative estimation of seismic landslide hazards. Most of the studies observed uncertainty regarding the earthquake scenario.

The improvised fully probabilistic approach (Alexey et al., 2020) has been applied in the present study to estimate the consistent earthquake scenario for seismic slope stability. The method handles the uncertainties in the data and provides reasonable hazard management. The framework of the approach aims to find the total probability of the slope failure under various ground-shaking levels (Del Gaudio et al., 2003). Some researchers have used this approach to develop the annual frequency of exceedance for the given sliding displacements (Rathje etal., 2008; Martino et al., 2019; Del Gaudio et al., 2003).

The present study aims to examine the impact of the Tindharia landslide, which occurred because of the September 18, 2011 earthquake and destroyed the World Heritage Site in the area. The landslide occurred in seismic zone IV in the Darjeeling region in West Bengal, India. This zone is highly vulnerable to earthquakes and is seismically active due to many seismic sources. Thus, considering the suitable selection of the earthquake scenario is important for the seismically induced landslide hazard assessment.

2. Methodology:

Fully probabilistic approach

A fully probabilistic approach represents the entire probability chain of the seismically induced landslide from strong motion prediction to mode of deformation. The approach accounts for two essential stages of calculation: evaluation of probability of occurrence of various PGA (y_i) within a certain period and determination of conditional probability at which the landslide triggers a given PGA. The total probability of the slope in the next T years is calculated using the following equation:

$$P_T(\text{slope failure}) = \sum_j \sum_i \mathrm{w_j P_T(PGA = y_i).P(slope\ failure|\ y_i, model\ j)} = \sum_j \sum_i \mathrm{w_j p_{ij}}, \qquad (1)$$

where P_T (PGA = y_i) occurrence probability of PGA (y_i) in a certain time interval and P(SF| y_i, model j) is the probability of the slope failure under seismic loading (y_i) for slope model j. Geo-mechanical models of slope were ranked by weight w_j, where $\sum_j w_j = 1$.

Probability of occurrence of PGA

PSHA is involved in the development of seismic hazard curves to address engineering safety issues in specific hazard levels (Raghukanth and Iyengar, 2007). The main goal of the analysis is to determine the probability of exceedance of a particular PGA in specified time intervals of seismic hazard curves (Cornell, 1968). The analysis is based on all possible sources in the site with all possible earthquake magnitudes, site-to-source distance, and GMPE. The calculation of all the sources that exceed the acceleration a is

$$\lambda(\mathrm{PGA} > y) = \sum_{i=1}^{n_{\mathrm{sources}}} v(\mathrm{M_i} > m_{\min}) \sum_{j=1}^{n_M} \sum_{j=1}^{n_R} \mathrm{P}\big(\mathrm{PGA} > \mathrm{y_i} \mid \mathrm{m_j}, \mathrm{r_j}\big).\, \mathrm{P}(\mathrm{M_i} = \mathrm{m_k})\,.\,\mathrm{P}(\mathrm{R_i} = \mathrm{r_k}), \tag{2}$$

Where, $n_{sources}$ represent the potential earthquake sources, and n_M and n_R represent the number of possible earthquakes and distances. $P(M_i = m_k)$ and $P(R_i = r_k)$ are the probability of magnitudes and distances in source i. v, the average rate of the threshold magnitude greater than the minimum magnitude, can be expressed as

$$v = 10^{a-bm_o} \tag{3}$$

Where a and b parameters are constants and m_o is constant mean annual rate of exceedance. These three parameters are obtained from the EC using Gutenberg–Richter distribution.
The probability of magnitude is

$$F_M(m) = \frac{1-10^{-b(m-m_0)}}{1-10^{-b(m_{max}-m_0)}}, \tag{4}$$

Where $F_M(m)$ is the cumulative distribution function and m_{max} is the maximum magnitude that the source produces.

The $(PGA>y_i| m_j, r_k)$ is the probability of exceedance of the PGA for acceleration y_i for m_j and r_k. The probability of exceedance of any PGA value is derived as follows:

$$\mathrm{P}(\mathrm{PGA} > y \mid \mathrm{m}, \mathrm{r}) = 1 - \phi\left(\frac{\ln(\mathrm{y}) - \ln(\mathrm{PGA})}{\sigma_{\ln\mathrm{PGA}}}\right) \tag{5}$$

Where, σ_{lnPGA} is the standard deviation.

The probability of exceeding the PGA value (y_i) in the next T years is

$$\mathrm{P_T}(\mathrm{PGA} > y) = 1 - \mathrm{e}^{-\lambda(\mathrm{PGA}>y).\mathrm{T}} \tag{6}$$

The probability of occurrence of a discrete set of ground motions is as follows:

$$\mathrm{P_T(PGA = y_i) = \; P_T(PGA > y_i) - P_T(PGA > y_{i+1})} \tag{7}$$

Equation (7) is used to evaluate the total probability of slope failure in equation 1.

Conditional probability of Tindharia landslide

The second step in calculating the fully probabilistic analysis is to know the probability of slope failure under seismic loading. The analysis is evaluated using Jibson probabilistic model (Jibson et al., 2000), which corresponds to Weibull distribution shown in the equation 8 The model calibrated with predicted sliding displacement (D_N) in cm, critical acceleration (a_c) and peak ground acceleration (y) based on Newmark approach (Newmark, 1965). The Newmarks approach assess the probability of slope triggering given the critical slope acceleration (a_c) and PGA value (y).

$$P(slopefailure|D_N) = 0.335[1 - \exp\left(-0.048{D_N}^{1.565}\right) \tag{8}$$

Where,

$$logD_N(y) = 0.215 + [\left(1 - \frac{a_c}{y}\right)^{2.341} \left(\frac{a_c}{y}\right)^{-1.438}\Big] \pm \; 0.51 \tag{9}$$

Many empirical relations are combined with Newmark's displacement (D_N) and intensity. However, in the present study, the predicted Newmark's displacement of the slope is evaluated with PGA using the above equation 9 (Romeo, 2000).

The critical acceleration (a_c) is a function of slope geometry and static factor of safety (F_s) and given as

$$a_c = (F_s - 1)gsin\alpha \tag{10}$$

Where, F_s and g are the static factor of safety and factor of gravity, and α is the dip angle of the sliding surface.

The static factor of safety is calculated using simplified limit equilibrium model of an infinite slope under certain assumptions based on Newmark approach (1965). As per the Newmarks approach, when some internal deformation accumulates the sliding mass the failure of landslide starts. When the seismic acceleration exceeds the critical value the accumulation of inner deformations takes place.

The approach considers the landslide mass sliding along planar surface. The assumptions in the model considered are as follows: the slope is homogeneous, the effect of pore pressure is negligible, the static safety factor is stress independent (constant), the sliding mass of the slope is rigid solid, and coefficients of static and dynamic friction are equal and constant. The static factor of safety (F_s) according to limit equilibrium theory is given as follows (Jibson et al., 2000):

$$F_s = \frac{c'}{yzsin\alpha} + \frac{tan\varphi}{tan\alpha} + \frac{my_w tan\varphi}{ytan\alpha} \quad (11)$$

where c' is cohesion, φ is friction angle, z is slope normal thickness, γ and γ_w are unit weight of material and ground water, α is dip angle of the sliding surface, and m is the saturated sliding mass thickness.

The soils in the area are saturated most of the year, so pore pressures are neglected from the equation 11 and paid great attention on third term of the equation.

In the next T years, the total probability of seismically induced landslide is obtained by substituting equations (8) and (7) in equation (1).

3. Details of study area:

The Tindharia slide is located at latitude and longitude of $26°51^{1}14.55^{11}$N–$88°20^{1}13.12^{11}$E in Darjeeling hills, West Bengal, India. The landslide is beneath the century-old Darjeeling toy train used in tourism. The slope failed on September 18, 2011 after the earthquake in Sikkim, Nepal (Figure 1).

(a) (b)

Picture 1 Aerial photographs of study area: (a) front view and (b) side view
(Source: Save the Hills)

The triangular debris slide was trigged initially by the earthquake, and the debris is widely spread over the entire site and deposited at the lower part of the slope. The upper part of the slope consists of colluviums and residual soils with varying thickness of 0–8m after the earthquake. After the initial earthquake-induced slope failure, the destabilization and series of failures were observed in the study area because of heavy rainfall on

September 28th, 2011. The debris had been eroded and washed away due to surface runoff. Highly weathered sandstone was exposed in some areas on the slope, revealing open cracks, especially at the top of the slope. Furthermore, destabilization and failure at the toe of the slope was observed due to stream erosion. A stream of water flowed toward the bottom of the slide and played a vital role in mass wasting and initiating further instability on the slope.

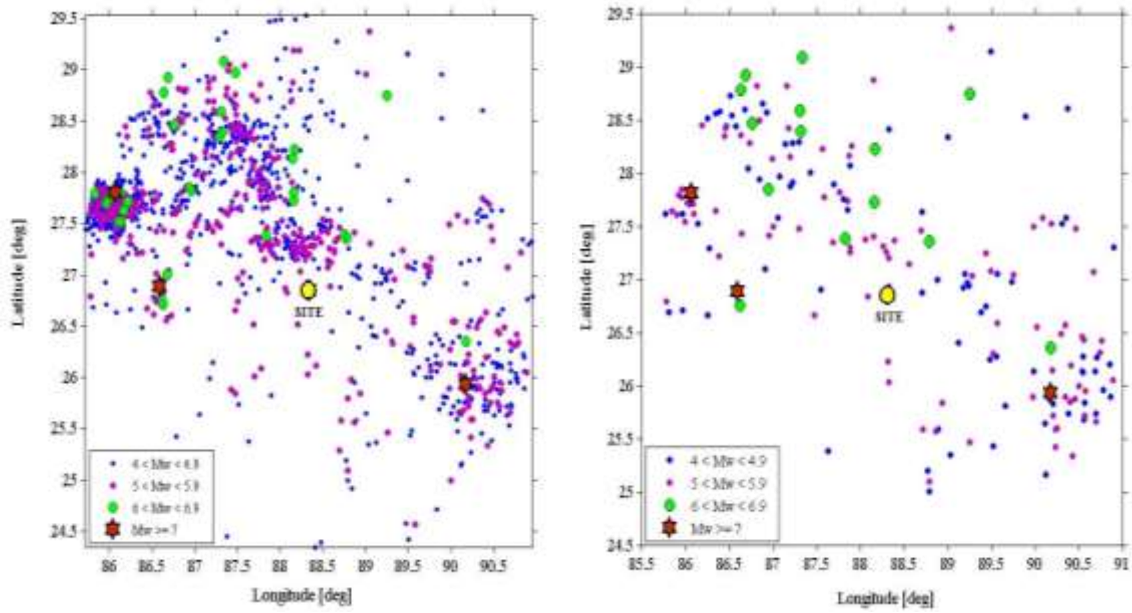
Material properties

The Tindharia landslide altered at elevation ranges of 600–800m with 30°–45°slope. According to the geological profile, the top layer of the study area is covered with residual soils and colluviums mainly with coaly shale and sandstone from the Gondwana group. A coal band is observed at the toe of the slope. The bottom layer of the slope is covered with highly weathered sandstone. The slope parameters for the hazard assessment were selected from the geological report of the study area by Kundu (2019).The soil parameter cohesion (c') and friction angle (φ) are 7.8 Kpa and 38°.The unit weight of material (γ) and groundwater (γ_w) is 19 and 9.8kN/m3. The slope normal thickness (z) and dip angle (α) are 4m and 28°. The saturated sliding mass thickness (m) is considered as 1 in the present study.

4. Preparation of EC and seismo-tectonic map:

EC preparation is the most fundamental step in PSHA. To prepare the EC, the following procedure is conducted: collection of earthquake data, homogenization of earthquake magnitude, de-clustering of the catalogue, and checking for data completeness.
The present study area is in the Bengal basin, which was seismically stable before 1930 and vulnerable after several seismic sources around the site produced remarkable ground motions. At a radius of approximately 300km from the site, magnitudes in the range of 4.0–8.0 were collected from 1932 to 2019 (91 years) and used in the present analysis. The region included active thrusts, faults, and lineaments. A total of 1,227 point sources along with 21 potential linear sources before declustering are collected along with magnitude scales, focal depth, time, and date. The point sources are collected from historical and instrumental records (USGS, ISC, and IRIS) and published literature. The linear sources are collected from the Seismotectonic Atlas of India (SEISAT 2000). The collected earthquake data are in different magnitude scales, so the homogenization of the collected data to one moment magnitude scale was conducted based on the empirical relations presented by Scordilis (2006) and Deniz and Yucemen (2010).

The de-clustering is necessary because the earthquake events collected from various sources are the raw data with good possibility of dependent events such as foreshocks and aftershocks. Furthermore, as we collected the data from various sources, the same events that were repeated with the same magnitude or with slightly different magnitudes were observed carefully and removed. The aftershocks and foreshocks from the homogenization catalogue were de-clustered based on the window method of Gardner and Knopoff (1974) using ZMAP software (Wiemer, 2001). After the removal of all independent events, out of 1277 events a total of 189 main shock events (84.64% of total records) were in the final catalogue. The before and after de-clustering of the events for the present study are shown in the Figure 2 (a & b).

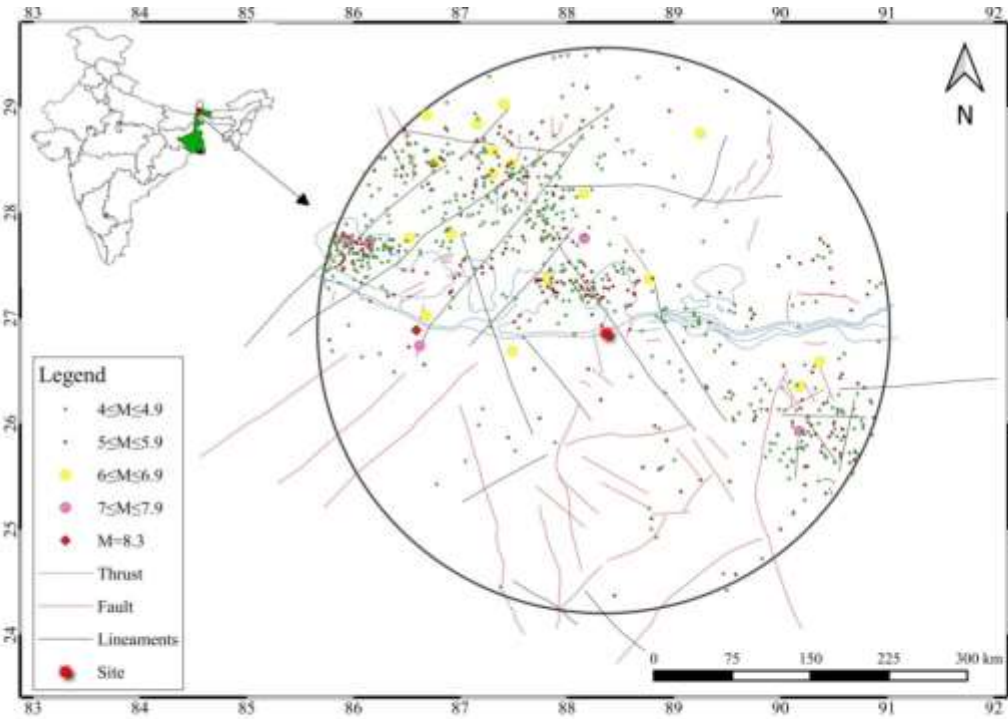


Picture 2 (a) Before and (b) after de-clustering of catalogue.

The complete catalogue with moment magnitudes of Mw>4 is summarized in Table 1. The seismo-tectonic map for the present study area is shown in Figure 3.

Table 1
Summary of final earthquake catalogue

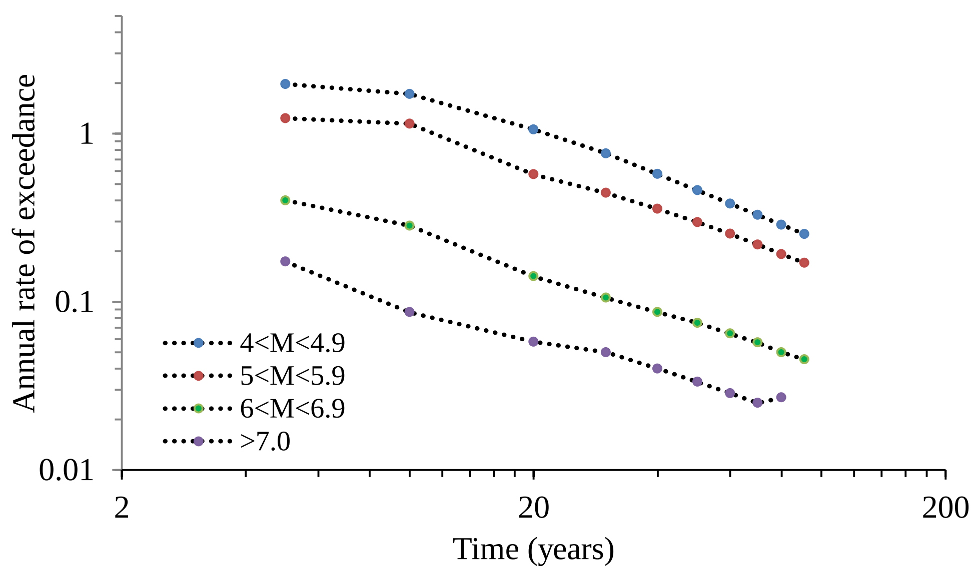
SI No.	Moment magnitude (Mw)	Number of events
1	$4 \leq M_w < 4.5$	17
2	$4.5 \leq M_w < 5$	60
3	$5 \leq M_w < 5.5$	60
4	$5.5 \leq M_w < 6$	33
5	$6 \leq M_w < 6.5$	10
6	$6.5 \leq M_w < 7$	5
7	$M_w \geq 7$	3



Picture 3 Location of site along with seismic tectonic features and distribution of past earthquakes (1932 to 2019) with in 300 km radius.

5. Analysis of Results and Discussion:

Data checking for completeness in terms of quality and quantity to ensure reliable results was conducted using a statistical test proposed by Stepp (1973) as shown in Figure 4.



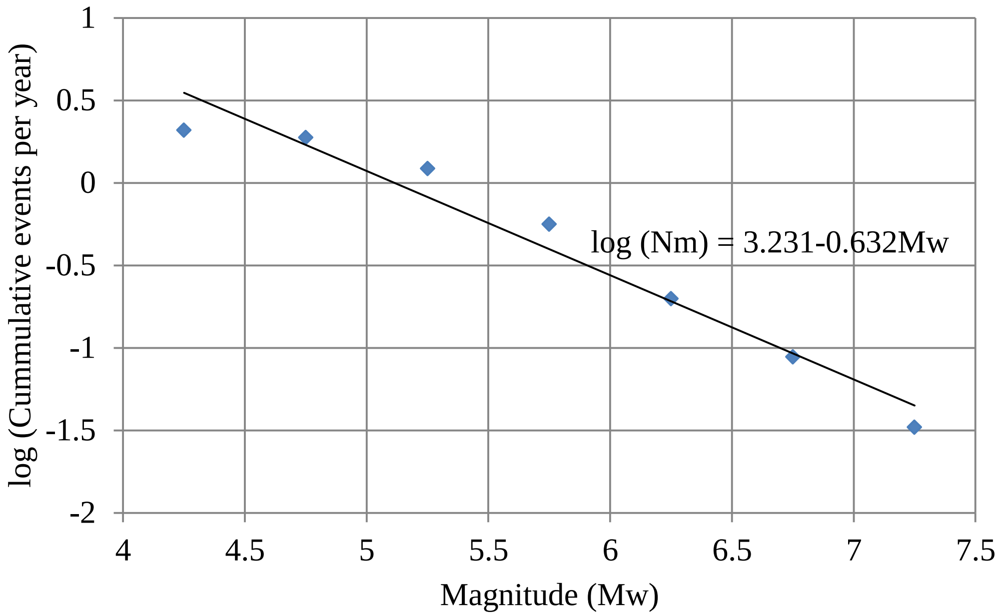
Picture 4 Data completeness based on Stepp’s method.

The seismicity parameters were evaluated using Gutenberg and Richter (G–R) model (1944) for the improved EC. The distribution of the earthquake frequency in a region with respect to magnitude is only frequency magnitude distribution estimated using the G–Rrecurrence model expressed as

$$\log \lambda_m = a - b M_w, \tag{12}$$

where λ_m indicates the cumulative number of earthquakes with magnitudes greater than or equal to M_w. The 'a' parameter describes the background seismicity, i.e., mean yearly number of earthquakes in a region, and 'b' describes the relative ratio of larger shocks to smaller shocks.

From the seismic parameters, the level of seismicity of the region can be evaluated. Thus, the G–R regional recurrence relationship for the study area is presented in Figure 5. The magnitude of completeness (Mc) is minimum magnitude event, where 100% is detected (Rydelek and Sacks, 1989) as estimated by Wiemer and Wyss (2000) method using the ZMAP tool (Wiemer, 2001). The site is divided into (0.1° * 0.1°) grid size and Mc is evaluated at the center of each grid within 300km radius.



Picture 5 Frequency magnitude distribution relation of entire study area.

In the present study, the seismic parameters 'a' and 'b' are estimated to be 3.228 and 0.631. The seismic parameters obtained are comparable with those from previous studies conducted in the same region.

GMPE

The 189 seismic point sources within 300km radius from the site are distributed by one or the other nearest dynamic potential active tectonic features such as thrust, faults, and lineaments. Thus, in the present study, the 20 line-source combinations of active faults, lineaments, and thrusts were selected for seismic hazard assessment of the study region (Table 1). The different hypocentral depth is considered within the range of 1–70 km depending on the faults for real hazard estimation.

Table 2
Details of seismic sources

SI.No	Fault Name	Shortest distance from site (km) (Rmin)	Mmax
1	East Patna Fault	190.478	7.1
2	Munger Sahastra Ridge Fault	130.129	7.2
3	Munger Sahastra Ridge	156.888	7
4	Rajmahal Fault	197.761	6.7
5	Malda Kishanganj Fault	79.037	5.3
6	Jangipur Fault	244.137	7.2
7	Gaibandha Fault	199.347	7.2
8	Debagram Bogra Fault	256.412	4.4
9	Dhubri Fault	171.615	7.8
10	Katihar Nailphamuri Fault	105.396	7
11	West Patna Fault	243.126	7.2
12	Sainthia Bahman Fault	229.488	6.4
13	GouriShankar Lineament	229.487	7
14	Everest Lineament	183.12	6.6
15	Arun Lineament	136.49	7.2
16	Kanchenjunga Lineament	89.844	5.5
17	Purnea Everest Lineament	99.734	5.7
18	Tista Lineament	10.981	7.1
19	Main Boundary Thrust	4.321	7.3
20	Main Central Thrust	8.214	8

Many researchers have developed different GMPEs for various regions of India depending on the observed and available datasets to estimate PGA. The GMPE selection among several available GMPEs is an important step because it greatly influences the final hazard assessment. The present study area consists of two zones: Bengal basin zone and northeastern Himalayan zone. The range of shear wave velocity is ranging from 100 to 3800 m/sec for Bengal Basin (Mitra et al. 2008 and Nath et al., 2010). The tectonic features of the study area are mostly influenced by those of the most active Himalayan region. Few GMPEs have been developed for the study area, but the best suited GMPE was checked and selected for the present study area is of Anbazhagan et al. (2019) which is performed with magnitude range Mw 4-9, hypo central distance range 10-750kms and shear wave velocity V_{s30} of 2000m/sec; and is expressed as follows:

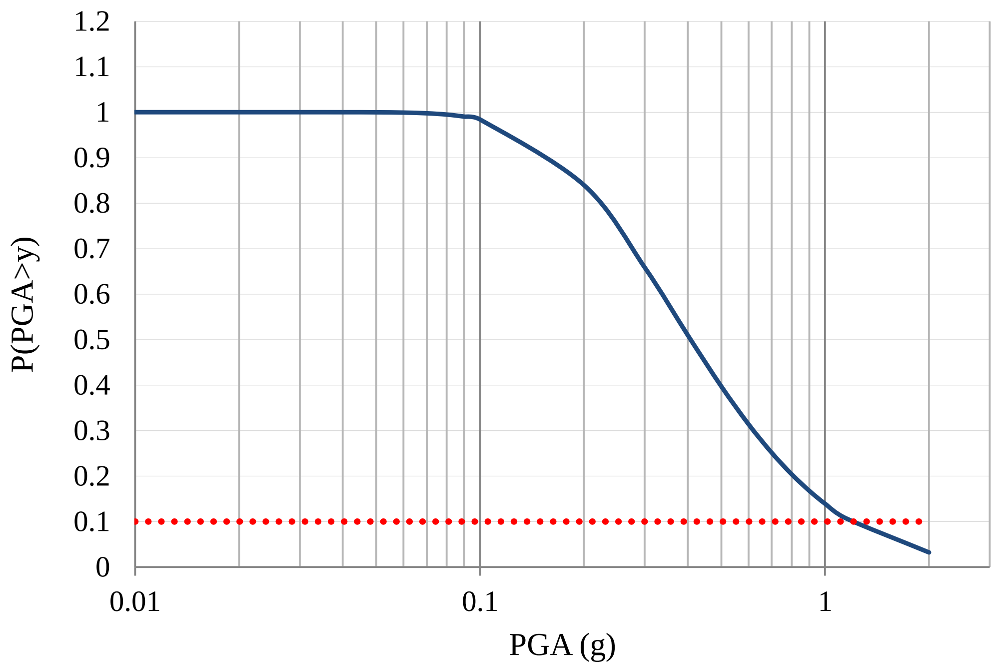
$$\ln(Y) = a_1 + a_2(M-6) + a_3(9-M)^2 + a_4 \ln R + a_m \ln\ R(M-6) + a_7 R + \sigma \quad (12)$$

$a_m\ = a_5,\ (M_w < 6.0\ and\ R < 300)$ or else is equal to a_6,

Where Y is the ground motion; a1, a2, a3, a4, am, a6, and a7 are the regression coefficients; M is the moment magnitude; R is the hypocentral distance; and σ is the standard deviation.

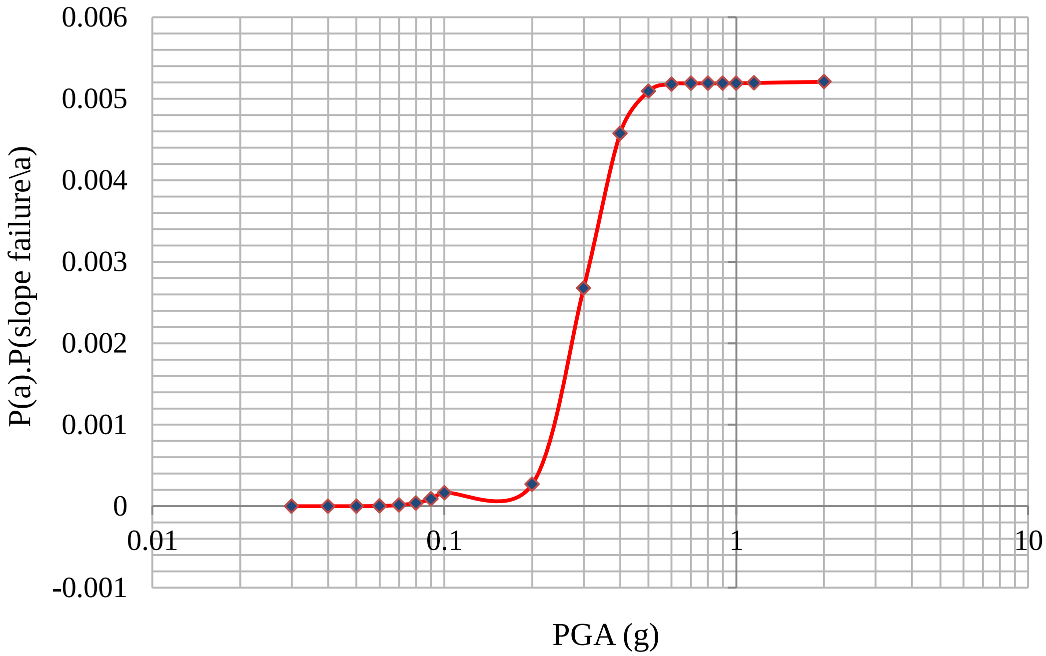
6. Results and discussion:

The PSHA study is performed for the evaluation of seismic hazard curves, which represent the PGA against the mean annual rate of exceedance. The seismic hazard curve shown in Figure 6 is computed using CRISIS (2007) software. The design PGA from the seismic hazard curve corresponding to 10% probability of exceedance (475-year return period) for the study area obtained is 1.02g. The PGA obtained from the seismic hazard curve is compared with the previous research result in the study area. The obtained PGA shows a good match with that provided by previous studies.



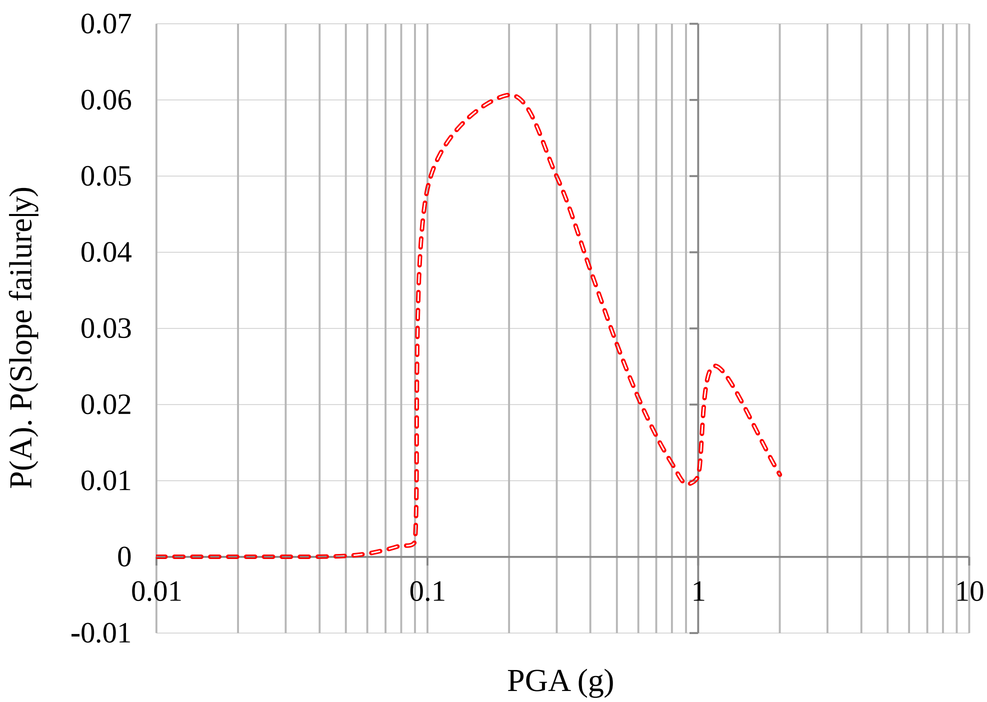
Picture 6 Cumulative hazard curve of 20 potential sources.10% exceedance probability level in 475-year return period is indicated in red dotted lines.

The critical acceleration (a_c) evaluated for the present slope model using Newmark's approach is 0.022g (equation 5). The obtained critical value (a_c) is substituted in the Jibson probabilistic model (equation 4), which provides the probability of landslide occurrence (Pr (DN(y_i)) in relation to ground shaking level (y_i). The conditional probability of slope failure (P_{ij}) in the next 50 years with respect to PGA is shown in Figure 7. The total probability of occurrence of the earthquake-induced landside in the next 50 years is 30%.



Picture 7 Total probability in next 50 years (cumulative distribution).

The (P_{ij}) obtained was multiplied by available ground scenario (a_i) for the slope model. For the considered slope model of critical acceleration (a_c), 0.022g of the PGA observed is approximately0.1–0.3g (Figure 8). The far variation in this PGA was observed for the 475-year probability.



Picture 8 PDF of slope failure in next 50years

The PGA rates obtained from the 475-year shaking map intensity and fully probabilistic approach are 1.02g and 0.12g. Thus, the sliding displacements obtained from PSHA are much higher than those obtained by the fully probabilistic approach. The hazard deaggregation for estimating the most probable magnitude and distance from site to source has been performed in CRISIS software. The most probable earthquake scenario that would trigger the landslide in the next 50 years is M_w = 6.53 at a distance of R = 58km.

Conclusion

Ground shaking is an important factor in earthquake disasters. The appropriate ground motion is the key input in implementing risk mitigation measures and slope design. For the hazard assessment in the site, the improvised fully probabilistic technique is considered. The method is the multi-stage hazard approach that includes data selection, probabilistic seismic hazard analysis, geological investigations, and landslide probability calibration model.

The method is applied considering a Tindharia slope that is known asa seismically active slide from the past. A30% total probability of slope failure is observed in the next 50 years. The most probable seismic event obtained from fully probabilistic technique that would trigger the landslide in the next 50 years is PGA of 0.12g,M_w = 6.53, and R = 58km. The peak ground intensity predicted for the next 50 years from the probabilistic seismic hazard analysis is 1.02g. The corresponding Newmark's displacements predicted from the fully probabilistic approach are almost four times less than the sliding displacement obtained from the PGA of the 475-year return period.

The results depicted from the present study show that the significant difference in ground-shaking intensity is observed between the two methods. The landslide hazard is overestimated when considering the 475-year seismic hazard map obtained from the PSHA analysis. The fully probabilistic approach can handle all possible ground-motion scenarios and provide reasonable hazard management.

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Pull-out test of rock bolts: Analyzing the causes of failure

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Abstract

To stabilize the roof and walls of the tunnels/caverns, tensioned rock bolts (Mechanical/Cement or Resin grouted end anchorage type) are the modern support system used worldwide. These rock bolts (post-stressed active anchors) installed in pattern create a stress field around the excavated area of tunnel. The effectiveness of these rock bolts are evaluated through the well-known 'Pull-out test' conducted as per IS 11309:1985 guidelines, wherein a rock bolt is considered to pass the test, if it registers a displacement of ≤40mm with designated load applied. While conducting pull-out test, failure of the rock bolts is generally attributed in the order of assertion to - (a) poor ground/geology, (b) poor material quality of rods, but hardly attributing to (c) improper testing methodology and/or (d) poor workmanship. This paper discusses non-geological reasons of failure for Pull-out tests conducted on 28 nos. in-situ rock bolts in the Powerhouse Cavern, Pakal Dul HE Project, J&K (1000MW); out of which only 2 no. rock bolts passed. In this case, inapt testing method/approach i.e. inadequate torque application on the bolt (for the purpose of tightening/stressing); evincing ineffective end-anchorage at bolt head which caused failure of rock bolts during Pull-out-test.

Keywords: Rock bolt, Pull-out Test, Torque

1. Introduction:

Rock bolt typically is a rod having (mechanical/cement grouted/resin grouted end anchorage) and provisioned with a bearing plate and nut. It is a tensioned reinforcing element to be stressed immediately after installation by torqueing or jacking, by means of a calibrated stressing device. The rock bolt is synonym with active rock anchor. The instant case confers mechanical end anchorage type expansion shell rock bolts installed in Powerhouse Cavern, Pakal Dul HEP.

Pull-out test is the test of effective anchorage and bond strength between reinforcing element (bolt) & the rockmass housing the bolt. This test in tunnels/caverns are typically performed to assess the anchorage or pull-out capacity of rock bolts. The instant case dealing with mechanically anchored bolts, Pull out test is done in un-grouted bolts. Pull-out capacity becomes very much important because rock bolts are anchored into the stable rock mass located beyond the plastic zone/zone of failure. Rock bolts reinforce and mobilize the inherent strength of rockmass by offering confining pressure, increase the stiffness of the rock and contribute shear resistance to the rock joints. The rock bolts are tested by incremental loading until total extension of reinforcement from hole/rock face reaches 40mm or till it yields/fractures, whichever is early.

The maximum load generally applied on the rock bolts (expansion shell type) during pull-out test is 60% of Yield Strength of Bolt. The range of load for different bolt dia with corresponding Torque range is shown in Table-1 below.

Table 1
Range of Load & Torque for different Bolt dia

S. No.	Dia of rock bolt (mm)	Max. Load applied (Ton)	Corresponding torque (Newton meter or N-m)
1.	25	15	750
2.	32	24	1536
3.	36	30	2160

Pakaldul HE Project (1000MW) is located in Kishtwar district of Jammu and Kashmir state (UT) of India. It is a storage scheme on the river Marusudar, a tributary of Chenab. The project envisages construction of 184m high CFRD near village Drangdhuran, two nos. 9.6km long each having 7.2m dia HRTs and an underground power house (166m x 20.2m x 51m) located in the right bank hill of Dul Reservoir. Presently, project is under construction by CVPP Ltd.

By the end of Nov’2019, excavation of PH cavern for top heading part (from EL 1285 to 1278M) and benching up to EL 1259.5M was completed and further benching being in progress. The results of pull out tests conducted on 7.5 m long and 32Ø rock bolts installed in crown/top heading of power house cavern has been taken for this study.

2. Geological setup of Project area:

Geologically, the project area lies in inner Lesser Himalaya (Kishtwar window) under Kishtwar group of rocks. Dam complex & part of HRT lie in Kibar gneisses whereas power house complex is housed in Quartzite-Phyllite sequence of Dul formation.

As per geological data collected while excavation, the PH cavern heading zone encompasses fresh to slightly weathered, moderately jointed and strong quartzite with thin (<5cm) bands of weak phyllites at places. Rock mass is dissected by primarily 4 set of joints including sub horizontal hill side dipping foliations (355°-030°/15°-20°). No significant shear zone observed during excavation of the cavern. The rock class encountered in the central gullet as 35 % Class-II and 65% Class-III.

3. Rock supports installed in PH crown:

After completion of the central heading and side slashing, the crown of the cavern has been supported by 32Ø 7.5 m & 36Ø 9m long rock bolts (@ 3m c/c) & 200mm thick SFR shotcrete. The excavation drawing of central heading (From EL 1285M to 1278M) showing stages of excavation and details of rock supports is shown in sketch as under (Figure-1):

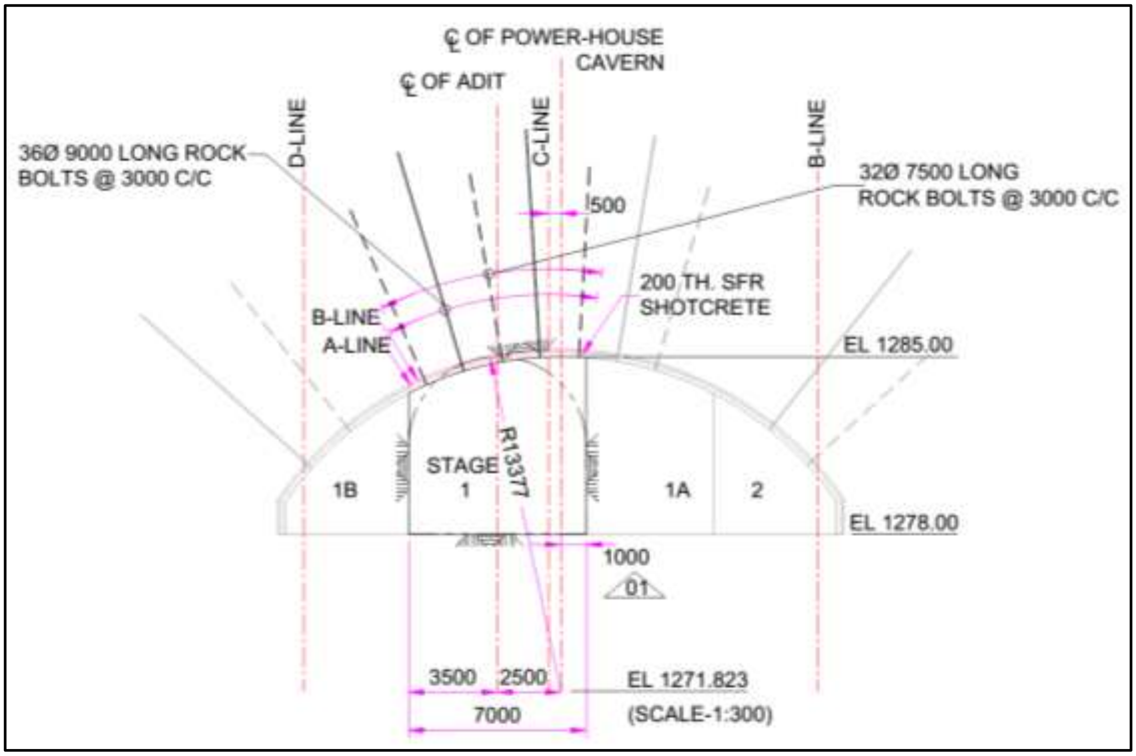


Figure 1 View of PH Cavern from Top Adit to PH Crown towards Main Access Tunnel

4. Pull out tests:

Subsequent to installation of desired 440 nos. (365 nos. of expansion shell type and 75 nos. resin capsule type) 7.5 m long, 32Ø rock bolts in the PH cavern (heading part, between EL 1278M to 1285m), **Pull-out tests** were carried out. It is to mention that Project had previously conducted pull-out tests in 28 no. rock bolts in heading part of cavern. Out of all the tested bolts, only 02 nos. expansion cell type rock bolts installed at RDs 114 & 123 side slash wall portion i.e. confining to Stage-1B & Stage-2 passed, showing less than 40mm displacement with the designated 24-ton load. Accordingly, few permutation & combination of bolt assemblies of different makes/brands were also tried by Project but registered failing results during pull-out test. Hence, it was apprehended by the Project & Contractor that poor geology is responsible for the failures of majority of the rock bolts.

In view of above, the geology encountered while excavation of PH cavern (heading part) was reviewed and it was observed that the rock mass in the entire structure area falls under fair to good category and devoid of any remarkable weak geological feature. So, to understand the real cause of Rock bolt failure vis-à-vis material property of bolt & its accessories/geology of the cavern or any other cause, two confirmatory pull-out tests with the details given here under were carried out in PH cavern area (Refer Table-2).

Table 2
Details of 2 Nos. Confirmatory Pull-out Tests in Powerhouse Cavern

Test No.	Location	Type of rock bolt	Load applied (Ton)	Displacement (mm)	Geology of the Testing location
1.	At RD 127m (R-8 hole-Right SPL, EL 1280M*)* in Stage-2 i.e. at rightmost side slashing wall heading portion of PH Cavern	32mm dia. 7.5m long Expansion shell rock bolt	4	6	Strong to very strong Quartzite with thin bands of Phyllite having Rock class III (Fair) with RMR value 58.
			8	12	
			12	18	
			16	24	
			20	32	
			24	**36**	
2.	At RD 126m (R-1 hole-Left SPL, EL 1279.8M*)* in Stage-1B i.e. at leftmost side slashing wall heading portion of PH Cavern		4	7	
			8	14	
			12	23	
			16	34	
			20	46	
			24	**53**	

Figure 2 Load Vs Displacement Graph for Confirmatory Pull-out Test No. 1&2
at RD 127m & 126m respectively

From the above Table-2 & Graph (Figure-2), it can be seen that despite being conducted in similar geological environment, the pull-out Test no. 1 got passed registering 36mm displacement with application of designated 24-ton load, whereas Test no. 2 failed registering 46mm displacement at 20 ton load and subsequent 53mm displacement with designated 24 ton load. From these test results, it could be conclusively inferred that local geology was not responsible for the failure of said pull-out tests conducted in PH cavern.

Further, to analyse the real cause of rock bolt failures, another confirmatory test (Test No.3) was done through re-tightening of nut & re-torqueing of the same bolt to go for another round of Pull-out test. Re-testing of already tested rock bolt i.e. R-1 hole, EL 1279.8m at RD 126 (Stage-1B) of PH cavern was carried out. Results of the same are shown as under in Table-3 & graphical representation of Load Vs Displacement in Figure-3, as under:

Table 3
Details of Another Confirmatory Pull-out Test (Test No.3) on Previously Tested Rock bolt at RD 126m (Stage-1B) in Powerhouse Cavern

Test No.	Location	Type of rock bolt	Load applied (Ton)	Displacement (mm)	Geology of the testing location
3.	At RD 126m (R-1 hole-at Left SPL, EL 1279.8M*)* in Stage-1B i.e leftmost side slashing wall heading portion of PH Cavern	32mm dia. 7.5m long Expansion shell rock bolt	4	2	Strong to very strong Quartzite with thin bands of Phyllite having Rock class III (Fair) with RMR value 58.
			8	4	
			12	8	
			16	10	
			20	12	
			24	14	
			26	16	
			28	21	

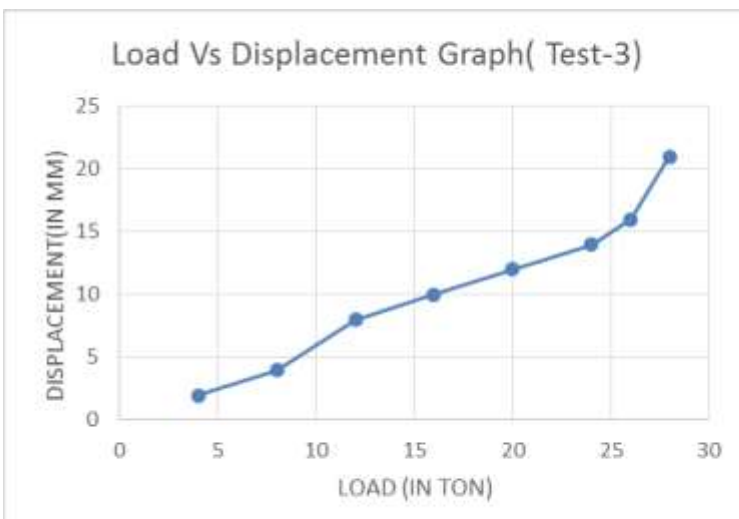


Figure 3 Load Vs Displacement Graph-Pull-out Test No.3 - RD126m (Stage-1B)

From the above observations in Table-3 & Figure-3, it could be understood that the rock bolt that had previously failed at 20ton load registering 46mm displacement, *after re-tightening* & *re-torqueing* showed passing value of 12mm displacement on the same load & 14mm displacement at designated 24 ton load for 32mm dia bolt (as per Technical specification). The results also indicate that these bolts could even take up further load beyond this designated 24ton load (60% of Yield strength of Fe 500 bolt) registering 21mm (<40mm) displacement at 28ton load during Pull-out test.

The above discussion indicates that probably proper torqueing was not done during pull-out testing of previous failed rock bolts. However, the 2 rock bolts (though under-torqued) that had previously passed prior to experimentation; successful pull-out result could be attributed to better bond strength of bolt (expansion shell) & rock due to higher competency and/or mere coincidence.

In this regard the range of torque provided by the testing personnel in previous tests were analysed and it was observed that the torque provided for 32mm dia rock bolts in the previous test was 226N-m against the desired torque of 1536 N-m (Refer Table-1). It means that the said rock bolt was torqued at quite lower value i.e. 15% of design stress value. Though the torque value was too less than the required, but during testing the bolt was being pulled out at its maximum/full designated stress value of 24 ton. Hence, it was apprehended that lower range of torque application might be causing failure of rock bolts.

5. Experimental Pull-out tests:

Now to validate this theory/working concept of "Designated value of Torque" for bolts to undergo stressing, prior to Pull-out test; the need for carrying out few additional validating experiments in PH cavern was felt. Accordingly, two experimental Pull-out tests were carried out with the available Torque wrench at site (Max. Capacity 540 N-m). The details of these experiments are mentioned as under.

5. (a) Experiment-1:

Experimental pull-out test was conducted with a better quality (SAIL make) 25mm dia, 3m long rock bolt available at site. Testing was done in similar geology at RD 125m (R-1 hole, EL 1279.6m) in Stage-1B (left side wall heading portion) of PH cavern i.e. near previously failed rock bolt which was installed and subjected to test (R-1 hole, EL 1279.8m at RD 126 of Stage-1B). This bolt was stressed with 540 N-m torque against the designated 750N-m. Subsequently, Pull-out test was done and results are given in Table-4 and corresponding Figure-4, as under.

Table 4
Details of Pull-out Test (Experiment-1) at RD 125m (Stage-1B) in PH Cavern

Load Applied (Ton)	Displacement (mm)
3	2
6	3
9	6
12	10
13	15

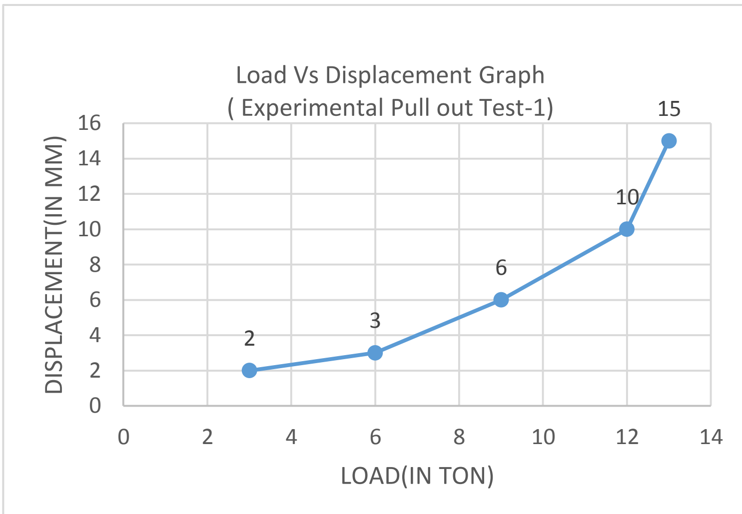
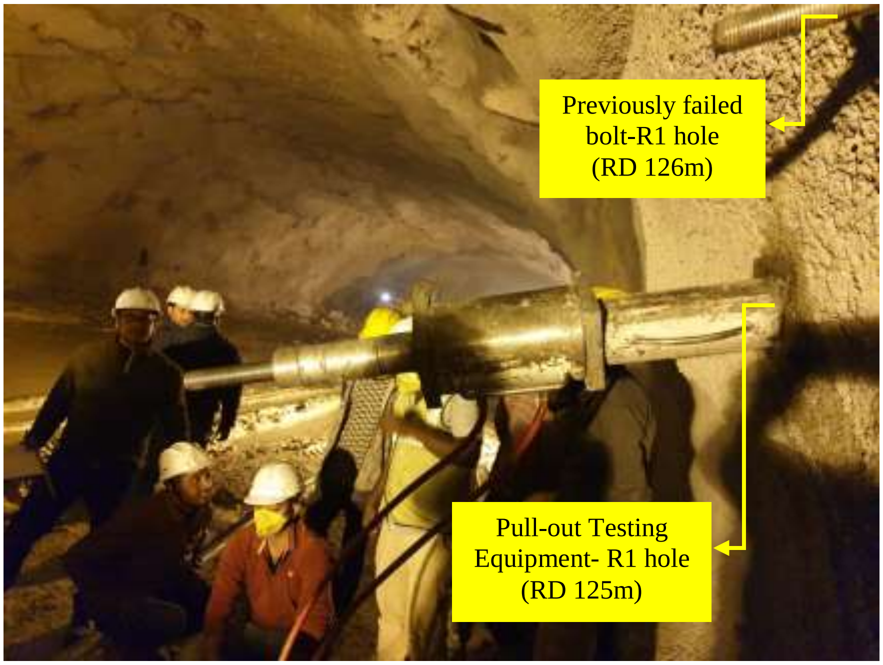


Figure 4 Load Vs Displacement Graph-Experiment-1 Pull-out Test
at RD 125m(Stage-1B) in Powerhouse Cavern

During testing, after applying 12 ton load, vibrations was observed in the testing element and subsequently when load increased up to 13 ton, there was a sound of slippage/breakage of rock bolt and could not take up further load. So, to see the mode of failure in the bolt; the bolt assembly was removed from the hole. It was observed that there was a slippage failure of shell/flange from the bolt thread.

From the above experiment, it could be clearly inferred that in similar geologic set up, the rock bolt (at R-1 hole, RD 125m) showed better statistics i.e. lesser displacement under higher torque application. It could easily take 13 ton load against designated 15 ton load and even registered very less displacement of 10mm at 12 ton load as compared to the bolt (at R1 hole, RD 126) that registered 23mm displacement at 12ton, which obviously had failed previously with lower torque application (Refer Test No.2 in Table-2). Further, it was observed that due to mechanical defect in the shell attached at bolt head, upon higher load application slippage failure occurred at bolt threading.

Pull-out test set-up for Experiment-1 at RD 125m (Stage-1B) in PH Cavern is shown in Picture-1 below:



Picture 1 Showing Pull-out Testing Equipment & Experiment-1 Pull-out at RD125m (R-1 hole, EL 1279.6m, Left SPL- Stage-1B heading portion of PH cavern) in sound rock mass–View from Top Adit to PH Crown towards Main Access Tunnel

5. (b) Experiment-2:

After satisfactory and better test result registered with Experiment-1 in sound rock mass, it was decided to do the same experiment in weaker rockmass (sheared/fractured zone) so as to understand the behaviour of expansion shell rock bolt with similar installation procedure, adopting similar testing methodology.

A thick sheared/fractured zone(±1m) was observed within Quartzite rockmass near Right SPL at RD132m. The rockmass being fractured, a smaller dia hole (38mm) was preferred therein for the experiment. Designated Hole dia being 38mm for 25mm dia bolt; Experiment-2 was planned to be conducted on a 25mm dia, 3m long rock bolt with its insertion in R-8 drill hole (Right SPL) at EL 1279M in Stage-2 (right side wall heading portion) of PH cavern. During insertion of the rock bolt jamming was experienced, probably due to partial collapse of hole or presence of drill cuttings due to drilling in fractured zone.

Similar as that of Experiment-1, this expansion shell rock bolt (25mm dia, 3m long) was stressed with the new Torque wrench made available at site thereby applying Torque value of 540N-m subsequent to opening of the shell at bolt head inside the hole bottom. Thereafter, Pull-out test was done on this 25mm dia 3m long bolt in presence of Senior Officers & site representatives of Pakal Dul HEP, CVPP and Senior officials of M/s Afcons (Power House Contractor)-Refer Picture-2 shown below. The results of Pull-out test are shown below in Table-5&6 with their graphical presentations in Figure-5&6 respectively.

Picture 2 Showing Pull-out Testing Equipment & Experiment-2 (a,b) Pull-out at RD132m (R-8 hole, EL 1279, Right SPL- Stage-2 heading portion of PH cavern) in sheared & fractured weak rock mass–View from Top Adit to PH Crown towards Main Access Tunnel

Table 5
Details of Pull-out Test (Experiment-2a) at RD 132m (Stage-2) in PH Cavern

Load (Ton)	Displacement (mm)
3	3
6	7
7	27

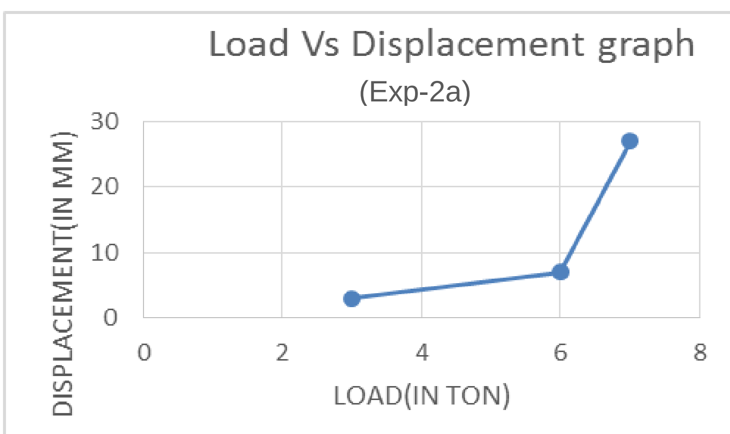


Figure 5 Load Vs Displacement Graph-Experiment-2a
Pull-out Test (1st attempt) at RD 132m(Stage-2) in Powerhouse Cavern

In the above Experiment-2a, the bolt couldn't take further load beyond 7 ton (Refer Table-5 & Figure-5). Hence, it called for re-testing (Experiment-2b) to check and verify the effectiveness of bolt. Results of Re-testing after releasing (zeroing) the hydraulic load in the jack are represented as under at Table-6 & corresponding Figure-6.

Table 6
Details of Pull-out Test (Experiment-2b) at RD 132m (Stage-2) in PH Cavern

Load (Ton)	Displacement (mm)
3	1
6	5
7	17
8	18

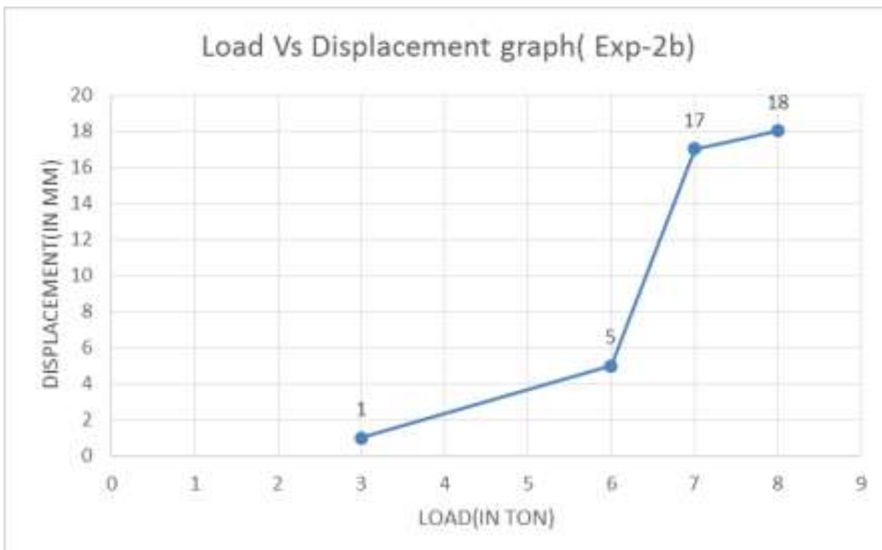


Figure 6 Load Vs Displacement Graph-Experiment-2b
Pull-out Test (2nd attempt) at RD 132m (Stage-2) in Powerhouse Cavern

Even in this second attempt (Experiment-2b, Table-6& Figure-6), Rock bolt couldn't take further load in Pull-out test beyond 7-8 ton, but showed better statistics with lesser displacement in such fractured rockmass as compared to Experiment-2a (Refer Table-5). Though effort was made to un-tighten the nut and remove the bolt from the hole to understand the failure mechanism, however couldn't retrieve the bolt due to jamming.

Nevertheless, it could be inferred that the Rock bolt could take Avg. 7.5 ton (i.e. 50% of the designated Load of 15 ton meant for 25mm dia) load registering an avg. satisfactory displacement value of ~18-27mm (<40mm) in weak (fractured) rockmass.

Above experiments (Experiment-2a & 2b) clearly show the role of proper/adequate torque application prior to pull-out testing of rock bolts that limits the displacement below 40mm as specified in IS 11309-1985. Further, it could also be inferred that geology should not be attributable for rock bolts' failure in pull-out tests executed in Power House Cavern heading portion, rather designated application of torque range is playing a key role.

The location plan of Rock bolts those have undergone Experimental-Pull-out testing at RDs 125,126,127 & 132 in PH cavern heading portion have been demarcated on 3D geological log, shown below at Figure-7.

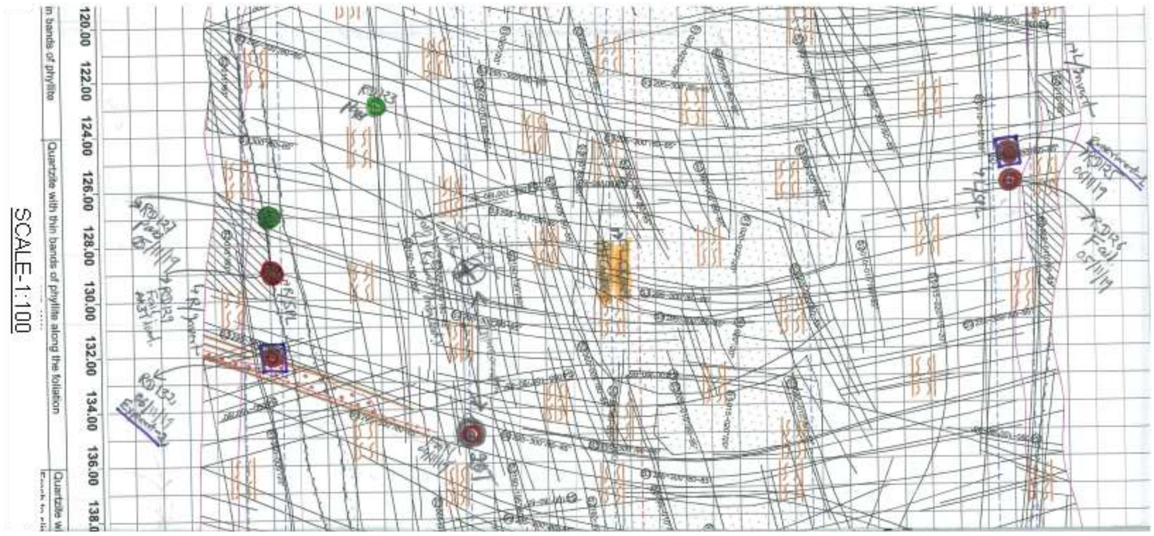


Figure 7 3D-Geological log of Power House Cavern Heading between EL 1285-1278m, showing location of Observed Confirmatory & Experimental Pull-out tests carried out on Expansion shell Rock bolts.

6. Project's Compliance & Final Results:

In view of the above experiments done in PH Cavern, conclusively inference was drawn in attributing under-capacity torqueing of rock bolts as the primary cause of failure during their pull-out tests. Accordingly, Project & Contractor were advised to avail 2 new calibrated Torque Wrenches with higher designated capacity (as per Torque Range mentioned in Table-1) on priority & re-torque the already installed un-grouted rock bolts with the same & re-install new bolts near the failed ones in compliance to Technical Specifications as dictated in Contracts. Consequent upon adoption of revised methodology at site with higher Torque range as per designated capacity, the first phase of pull-out tests done during Mid-November to 1st week of December 2019 showed remarkable results with passing of 30 bolts out of 34 re-torqued/newly installed bolts. The 4 no. bolts those still failed were due to end-anchorage flange & shell junction slippage that in-turn owed to material defect of threading in the bolt head (i.e. non-geological reason).

Subsequent compliance at site with higher-designated capacity torqueing of bolts & dedicated installations, further benching excavation in PH Cavern below heading (i.e.

below EL 1278m) could be successfully resumed at site after a considerable hold & delay.

7. Conclusion and Recommendations:

From the above discussed confirmatory Pull-out tests & Experimental Pull-out test results, it could be inferred that geology/rockmass condition has no such pronounced impact/attribute to failure of rock bolts during Pull-out tests conducted in heading part of Power house cavern. Rather, improper/under-torqueing of bolts after installation was the main causative factor behind the said failures.

As torqueing keeps the reinforcing element as well as surrounding rock in stressed condition, it was also observed that, upon further stressing, the bolt was able to take up higher load registering lower displacement value. Hence, the role of torqueing of rock bolt to its design stress/tension value prior to Pull-out test is imperative & needs to be done for effective support of underground structures. Further, it is pertinent to mention here that Project executing representatives need to ensure availability of Torque Wrench/testing equipment at site which should be capable of stressing the largest diameter of rock bolt to the yield stress of the bolt. Based on the experiments conducted, it is also suggested that stressing of bolts in fractured rockmass need to be done at lower stress value (less than 50% of design value), as observed during experimentation at site.

Conclusively, it can be remarked that testing of rock support elements, their behaviour (in addition to site geology), deficiencies (if any) in the material property, site executional procedures & technical know-how of stressing/testing equipment vis-à-vis methodology adopted at site; for rock support installation to be successful & effective, all these parameters need to be comprehensibly looked into instead of having stereotype approach at site.

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Role of geotechnical monitoring instrumentation in Tehri Pumped Storage Plant (1000 MW) – A case study

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Abstract

Instrumentation and monitoring are an essential part of current tunnelling practice. For safe and economical tunnelling in sensitive construction environments a continuous adaptation of excavation and support design is required so that input parameters can be revised when the predictions deviate from measured values. To understand the rock mass behaviour it is necessary to make a note of all parameters which needs to be measure in pre and post construction stages. Systematic monitoring results can provide valuable information pertaining to imminent collapse, thus making it possible to control the tunnel stability by providing proper counter measures. Excavation of any underground openings results in the release and readjustment of three dimensional stresses around the cavity. This results in displacements/deformations which are time dependent. Extensometer is used to measure deformation of a section of rock mass and adjacent surrounding strata with the help of anchors at different depths. The depth of anchors varies with type of rock strata and the location of fix point with respect to which deformation are to be measured. More importantly, it involves detailed planning to finalize the position of each monitoring instrument based on location and orientation of geological features. The present geotechnical monitoring practices for underground structures involve convergence monitoring with the help of optical targets and rock mass displacement with help of bore hole extensometers. In addition to this load cells are also used for recording and monitoring of loads in structural elements like rock bolts and cable anchors. This paper briefly describes about the deformation of rock mass in underground structure of Tehri Pumped Storage Plant for long term monitoring their results and conclusions.

Key Words: Underground Excavation, Geotechnical monitoring Instrumentation, Warning limits, MPBX, BRT and Load cells.

1. Introduction:

The geotechnical instrumentation has a vital role in evaluating the structural performance of an underground structure. The natural ground or rock mass tends to deform and de-stress when subjected to excavations, foundation and other loadings etc. Activities like squeezing, swelling and creep depend upon the mechanical characteristics of the material and are responsible for the disturbances inside the underground rock mass.

The long term performance of an underground structure is monitored by installing the structural instruments to predict and evaluate the safety of excavated openings. However, the question on number, type and locations of instruments can only be addressed by fortuitous combination of understanding of underground structural behaviour, experience and judgement. The instrumentation design therefore needs to be conceived with care and considerations of site specific conditions associated with the structures. (Kelllaway, M., Taylor, D. et.al.)

Various types of instruments are used for underground structures including system to record by key monitoring parameters listed as under:

1) Convergence – Deformations of excavated surface and surrounding rock mass
2) Load developed in rock bolts after installation and tensioning
3) Pore water pressure in the rock mass
4) Earth Pressure developed due to excavation

Multiple underground structures are under excavation at Tehri Pumped Storage Plant along with several adits, tunnels, shafts and cavern. The structural behaviour of these openings has been monitored by instruments installed at respective location along the length and cross section. Further, it is equally important to ensure proper recording of monitoring data in order to analyze and take immediate action in case of any adverse conditions observed.

2. Brief Description of Project:

As an integral part of Tehri Power House Complex located in the state of Uttarakhand in Northern India; an underground 4 X 250 MW Tehri Pumped Storage Plant is under construction based on the concept of upper and lower reservoir. The Tehri dam reservoir will function as the upper reservoir and Koteshwar reservoir as the lower balancing reservoir. On completion additional generating capacity 1000 MW (4 X 250 MW) peaking power will be added to Northern grid. During non-peak hours, water from lower reservoir would be pumped back to upper reservoir by utilizing the surplus available power in the grid.

Tehri Pumped Storage Plant comprises construction of Power House, two nos. of upstream Surge Shafts, Butterfly Valve Chamber, Penstock Assembly Chamber, two nos. of downstream Surge Shafts, four nos. of Penstocks, Bus bar Chamber, Ventilation tunnel, Drainage Galleries, two nos. Tail Race Tunnels and Outfall Structure located on the left bank of Bhagirathi River.

3. Geology of the area:

Tehri Project area lies within the Main Himalayan Block (MHB), in the midlands of Lesser Himalayas, bounded to the north and south by regional tectonic lineaments – the Main Central Thrust (MCT) and Main Boundary Fault (MBF) respectively. The former, to the north separates the meta-sedimentary sequence of Lesser Himalaya from the crystalline rocks of Higher Himalaya and the latter marks boundary between lesser Himalaya and tertiary sequence of Frontal Foothill Belt (FFB), in the south. The rock stratigraphy of lesser Himalaya exposed around the Tehri Project area are broadly classified into Garhwal, Shimla, Jaunsar, Bailana, Krol and Tal groups. The folded meta-sedimentary rocks exposed around the project site form an uninterrupted sequence of Chandpur Phyllites having a variable proportion of argillaceous and arenaceous constituents. Considering the rhythmicity of intercalated bands and varied the degree of tectonic effects in them, the Phyllites at project side have been classified into mainly four lithological variants as described below.

Phyllitic Quartzite Massive (PQM)
Phyllitic Quartzite Thinly Bedded (PQT)
Quartzite Phyllite (QP)
Sheared/Schistose Phyllite (SP)

PQM and PQT are more quartzite (arenaceous) and rarely micaceous in composition and are coarser in grain size. These rocks are grey, dark grey, brownish grey, greenish grey, greyish grey and green in colour. It is mainly comprised of quartz, feldspar and oriented leths of micaceous minerals. QP is more areno-argillaceous in composition, fine-grained and dark coloured. SP comprises of argillaceous and deformed variants of PQM and PQT rock, formed in sheared zone area which has weak rock mass characterististics.

4. Instrumentation and Monitoring:

The main objective of instrumentation and monitoring are very important for any underground structure. Observations recorded through instruments installed at various underground structures serves as guide for taking proactive remedial actions. If the underground excavation encounters known or unexpected major geological features such as fault, shear zone or a highly weathered rock mass, instrumentation can be used to monitor its behaviour. The use of proper instrument at appropriate time can give very valuable information, which can help to prevent a likely major mishap. This can provide early warning of many conditions that could contribute any failure mechanism.

Type and selection of monitoring instruments based on geotechnical aspects and surrounding rock mass. Various monitoring instruments installed in multiple adits, tunnels, shafts and chambers at different cross sections as per approved drawings and in consultation with designers.

1.1 Instrument Installed:

The monitoring instruments installed at various locations in the underground openings are listed in Table 1 along with general specifications.

Table 1
Specifications of Monitoring Instruments installed at Tehri Pumped Storage Plant

S No.	Instrument	Type	Parameter	Unit	Range	Resolution	Sign Convention
1	Bi-reflex targets	Optical	Deformation of Excavated Surface	mm	-	2.omm	"+ve" displacement indicate Divergence in cavern "-ve" displacement indicate Convergence in cavern
2	MPBX (2m, 5m, 10m & 15m or 20m)	Vibrating wire	Deformations of rock mass	mm	0.0 – 50 or ±25m	0.01 mm for sensors	"+ve" displacement indicate separation of anchors from wall "-ve" displacement indicate convergence of anchors towards wall
3	Load cells (in rock bolts)	Vibrating wire	Load developed in rock bolts	tons	30m	Accuracy 1%	"+ve" value indicates increase in rock bolt load (Due to convergence) "-ve" value indicates decrease in rock bolt load (due to loosening or far field convergence)
4	Load cells (in cable anchors)		Load developed in cable anchors		200m		

The technical aspects of monitoring instruments installed at various components of project are briefly described as follows:

a) Bi-Reflex Targets :

Bi-Reflex Targets consists of reflector plate mounted on a robust frame. The target has reflectors on both sides and is mounted on a universal joint such that it can be oriented in any direction as required. The target has cross mark to allow precise targeting. It is used along with the convergence bolt and break off point as shown in Figure1 below:

Figure 1 A Bi-Reflex Target

The targets are generally installed in cross sections of underground openings at required locations and monitor the absolute position of point on the excavated surface. For monitoring the displacements in different type of roc class five, three and one bi-reflex target installed at a given cross section of tunnel profile. Targets are designated as T1, T2, T3, T4 and T5. The absolute displacement of each target is determined as displacement vector in terms of its magnitude and direction. As the direction of all the appropriate coordinates (i.e. Northing and Easting) are selected to determine the displacement vectors. The magnitude of displacement vector between two consecutives date is plotted with respect to time or date to analyze the behaviour of target. The continuous vector diagram in plane of tunnel cross section is superimposed on the cross section of the opening to analyze the movement of tunnel boundary.

Rate of movement is calculated as ratio of displacement magnitude in mm and number of days elapsed since last date of measurement. All three layouts of installed BRT's are shown in Figure 2a, 2b and 2c respectively along with indicative displacement vectors.

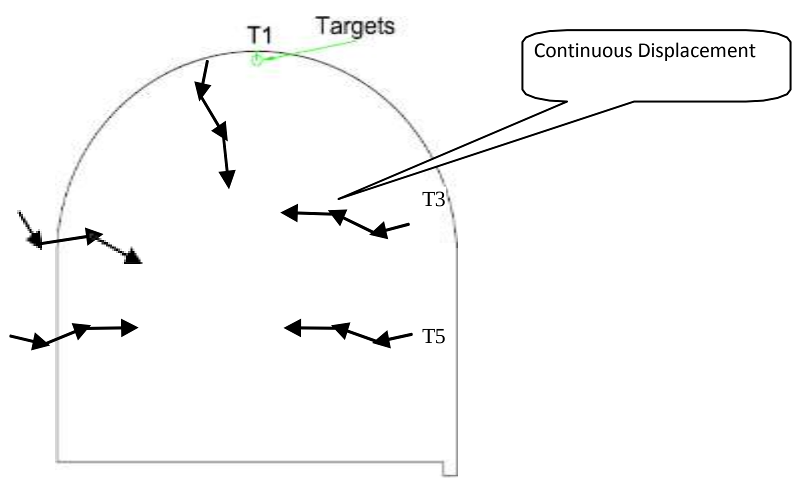

Figure 2a Five Target Section

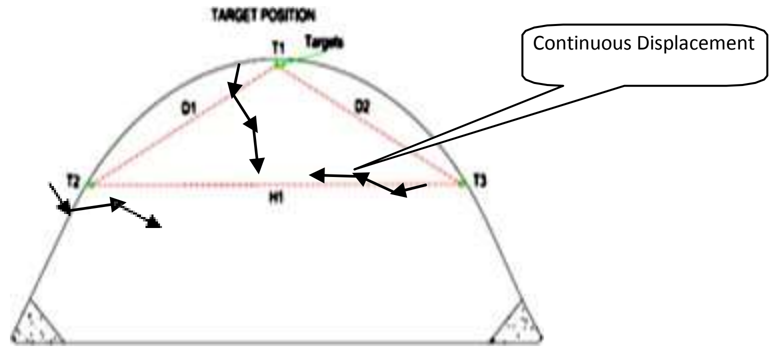

Figure 2b Three Target Section

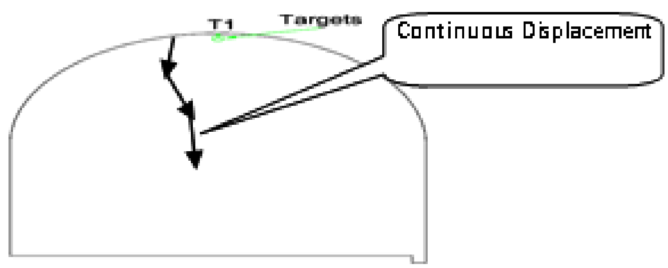

Figure 2c Single target section

b) MPBX-Multiple Point Bore hole Extensometer:

Multiple Point Bore hole Extensometer is used to measure the deformation of a section of rock mass with respect to deep anchor. MPBX can be either single point or multiple point consists of sliding rod, anchored at selected points within bore hole and fitted with vibrating wire sensors(Figure 3).The movement of the rock mass around bore hole is transferred to the attached anchors and same is recorded by transducers placed on the top of bore hole.

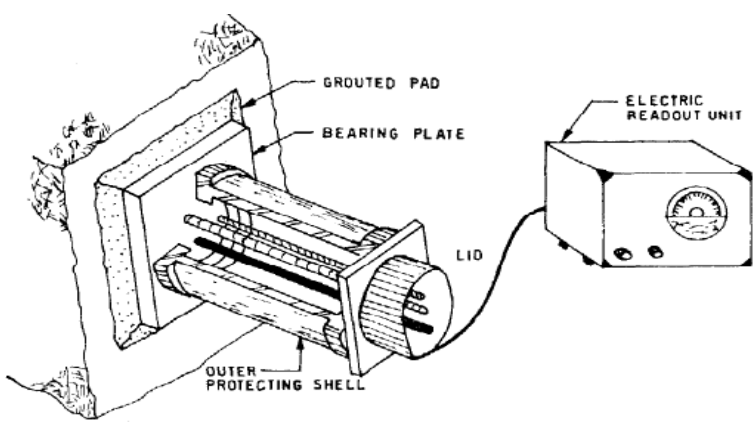

Figure 3 Collar Head at the Excavated Surface

The maximum number of anchors possible to install in a bore hole depends on diameter of bore hole and type of anchor used. The anchors can be attached to the surrounding rock mass in two ways; either by grouting the bore hole or by mechanical attaching with hydraulic anchors. For measurements of displacements the vibrating wire type sensors are used for long term measurements of relative displacements at four anchor points.

MPBX installed at Tehri Pumped Storage Plant has four anchor points (2m, 5m, 10m, 15m and 20m) are grouted type in which the anchors are grouted in to bore hole while keeping the movements transferring elements free with PVC pipe. Photograph of MPBX before and after installation shown in Figure 4 below:

Figure 4a Anchors

Figure 4b Rods with PVC Pipe

Figure 4c Collar Head

c) Load Cell in Rock bolts:

Vibrating wire load cells are used for recording and monitoring of loads in structural elements like rock bolts, tie backs, foundation anchors tunnel supports and in prestressing. It is installed at the time of the structural elements. A pretension force induced in the rock bolt recorded in the load cell. The distressing of the rock mass takes place due to any excavation or loading activity and the rock load is transferred to the rock bolts. The axial load which is developed in rock bolts is reflected in load cells as an increment to the initial installed load.

The vibrating wire load cell comprises of a set of three or six vibrating wire gauges, mounted parallel to each other, equally spaced in a ring in an alloy steel cylinder. The method of construction results in a very robust instrument suitable for use where high performance, longevity and mechanical strength are important. Typical arrangement of load cell installed along with rock bolt is shown in Figure 5.

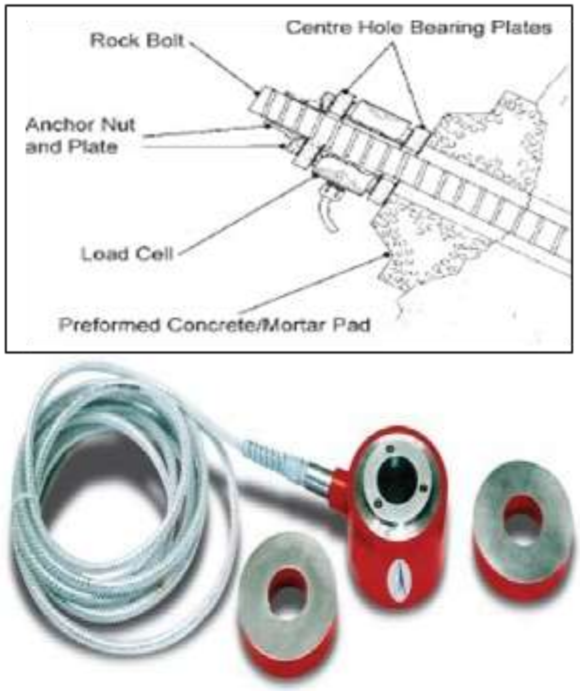

Figure 5 Load Cells and its typical arrangement with rock bolts

The monitoring of load cell data is also carried out periodically and the data obtained is analyzed with warning limits provided by designers. Generally the increase in rock bolt loads indicates the convergence in openings and vice versa.

1.2 Instrumentation Scheme:

Adequate instrumentation scheme has been specified by the design consultant, in particular the monitoring plan, frequency of readings, definition of threshold limits adopted in case of ground movements exceeding prescribed conservative 'trigger' limits are indicated. The general procedure for the implementation of countermeasures should be the movement exceeds the limit of displacement/stress established from design calculation for the structural integrity and safety of workers during excavation. Geotechnical monitoring instrument installed at various structure of project along with their excavated width listed in Table 2.

Table2
List of Underground Structure monitored by Geotechnical Instrument

S. No.	Structure Name	Structure ID	Orientation of opening w.r.t. North	Maximum excavated of opening (m)
1	Adit	AA-6	N30°	7.0
		AA-10	N08°	
		AA-11	N30°	
2	Butterfly Valve Chamber	BVC	N303°	10
3	Upper Penstock Assembly	PAC	N303°	13
4	Lower Penstock Assembly	LPAC	N71°	11.6
5	Tail Race Tunnel-3	TRT-3 D/S	N185°	11.0
6	Tail Race Tunnel-4	TRT-4 D/S	N185°	11.0
7	Ventilation Tunnel	VT	N102°	4.6
8	Bus Bar Chamber	BBC	N213°	10.0
9	Drainage Gallery	DG around BVC	N303°	4.4
10	D/S surge Shaft Chamber-3	D/S SSChamber-3	N30°	19.8
11	D/S Surge Shaft Chamber-4	D/S SSChamber-4	N30°	19.8
12	Link tunnel between D/S Surge Shaft Chamber-3&4	D/S SSChamber-3 & 4	N30°	11.5
13	Lower Bus bar Tunnel-5	LBB-5	N300-335°	7.9
14	Lower Bus bar Tunnel-6	LBB-6	N300-335°	7.9
15	Lower Bus bar Tunnel-7	LBB-7	N300-335°	7.9
16	Lower Bus bar Tunnel-8	LBB-8	N300-325°	7.9
17	Upper Bus bar Tunnel-5	UBB-5	N120°	7.9
18	Upper Bus bar Tunnel-6	UBB-6	N120°	7.9
19	Adit	AA-1	N54°-90°	7.0
		AA-2	N303°	7.0
		AA-3	N303°	7.0
20	U/S Surge Shaft Chamber-3	U/S SSChamber-3	Vertical Direction	15.0
21	U/S Surge Shaft Chamber-4	U/S SSChamber-4	Vertical Direction	15.0
22	U/S Surge Shaft-3	U/S Shaft-3	Vertical Direction	20.8
23	U/S Surge Shaft-4	U/S Shaft-4	Vertical Direction	20.8
24	Drainage Gallery around upstream Surge Shaft	DGUSSS	N325°	4.4
25	Power House	Service Bay	N30°	24.6
26		Crane Beam	N30°	24.6
27		Pipe Gallery	N210°	3.6

The type number and interval of readings are being finalized by the designer based on evaluation of previous monitoring results, trend and warning limits. The tunnel convergence monitoring is the predominant measure adopted for the all foreseen/ unforeseen roc mass condition. As earlier stated, the convergence readings are the main parameter for evaluating the stability of any underground structure, while other measures can be considered additional tool to know the behavior of rock mass.

1.3 Frequency of monitoring and warning limit:

The frequency of monitoring data is depended upon the warning limit of individual structure provided by designers in approved instrumentation drawing for individual project component. A pre determined or rate of change of a key indicator parameter that is considered to indicate a potential problem, but not of sufficient severity to require cessation of the works. Exceeding this trigger level will generally require a check on instrument function, visual inspection of structure being monitored, increase in monitoring frequency, review of the design and modification of construction process. The warning limit of different engineering parameters obtained from monitoring data to check the health of structure as under.

A) Deformation measurements:

Alarming limits for benching and heading activities for different structures obtained from approved instrumentation drawing and readings are taken in same reference line as mentioned in Table 3a and 3b respectively.

Table 3a
Max^{m} allowable and warning limit of deformation in underground openings of Tehri PSP

Structure	**Warning limit (MM)**			
	Heading		**Benching**	
	I level	**II level**	**I level**	**II level**
BVC (1st bench)	1% of the underground opening		50mm to 75mm	More Than 75mm
UPAC			50mm to 75mm	More Than 75mm
LPAC	1% of the underground opening			
PH	1% of the underground opening			
DSSS-3	50mm to 100mm	More than 100mm	150mm to 200mm	More than 200mm
DSSS-4	50mm to 100mm	More than 100mm	150mm to 200mm	More than 200mm
USSS-3	20mm to 40mm	More than 40mm	50mm to 60mm	More than 60mm
USSS-4	20mm to 40mm	More than 40mm	50mm to 60mm	More than 60mm
LINK TUNNEL	15	30	50mm to 100mm	More than 100
ADITS	1% of the underground opening			
DRAINAGE GALLERY	1% of the underground opening			
TRT-3	1% of the underground opening			
TRT-4	1% of the underground opening			
VENTELATION TUNNEL	1% of the underground opening			
TRT outfall	1% of the underground opening		30	40
DS SS	Rock Type- PQM + PQT PQT + QP QP + SP SP		 50 120 150 200	 100 150 200 250
US SS	Rock Type- Type-2 Type-3 Type-4		 50 100 200	 75 150 250

LOWER BUS BAR TUNNEL			100	150
UPPER HORIZONTAL PENSTOCK	Rock class II III IV V		30 50 100 200	
VERTICAL SHAFT	Rock class II III IV V		30 50 100 200	

Table 3b
Warning limit for rock bolt loads in UG Openings at PH Cavern of Tehri PSP

S. No.	Rock bolt dia. (mm)	Design load, D (Tons)	Warning limit W = 80 % of D (Tons)
1	25	20.0	16.0
2	32	35.0	28.0

B) Rock Load Measurements:

The warning limits for the rock bolt loads monitored by load cells installed in the rock bolts are suggested by 80% of the design capacity of the rock bolt. The design capacity of rock bolts are different for different size of bolts installed and so shall be the warning level for the rock load observed and recorded by instrument as shown in Table 3.

120 ton capacity of cable anchors locked at 80 ton installed in Power House, Butterfly Valve Chamber, Penstock Assembly Chamber and presently installation is in progress at the benches of Tail Race Tunnel outfall beyond El: 623.0m.

Regular monitoring data is taken based on the above warning limit considered for different instruments and the behavior of underground openings observed from the analysis of monitoring data in terms of correlation with major construction activities.

2. Regular Monitoring data processing and interpretation:

As stated earlier also monitoring data is regularly taken at site in consideration with their warning limits and frequencies for individual structures. The deformation occurred after the instrumentations are monitored regularly and interpretation work carried out accordingly on monthly basis. Maximum rate of displacement observed by Bi-reflex targets, MPBX and increment in load on rock bolts/cable anchors by load cells are summarized in Table 4, 5 and 6 respectively.

Table 4
Maximum Rate of displacement observed by Bi-reflex target

SNo.	Component	RD (m)	Class of Rock	Target ID	Maximum movement w.r.t. initial (mm)	Maximum Rate of Movement in last month (mm/month)
1	Adit AA-1	160	IV	T-3	34.60	0.00
2	AA-2 Drainage Gallery	25	IV	T-2	37.68	1.19
3	U/S Surge Shaft Chamber 3	16.3	IV	T-5	17.63	2.07
4	Chamber-3 Cracking	23.32	IV	Lower Left	12.17	0.00
5	USSS-3	EL: 802.80	III	OP-4	36.77	0.00
6	U/S Surge Shaft Chamber 4	2.2	IV	T-6	45.89	0.0
7	USSS-4	EL:802	III	OP-8	67.82	12.38
8	BVC	35.6m	VA	T-4	133.5	0.00
9	DG-USS	82.00	IV	T-2	40.59	0.00
10	PAC	30.32	IV	T-2	49.75	0.00
1	PH	41.5	IV	T-2	34.4	0.00
2	UDG-PH	0.00	III	T-1	40.51	0.00
3	PH Service Bay	16.34	IV	Left OP-1	15.5	0.00
4	LBB-5	10	III	OP-1	17.09	0.00
5	LBB-6	15	III	T-3	25.67	0.00
6	LBB-7	35	III	T-3	10.8	Target damaged in Jan'17
7	LBB-8	18	III	T-1	10.4	Target damaged in August'17
8	UBB-5	7.5	III	T-2	41.7	0.00
9	UBB-6	7.5	III	T-2	7.5	0.00
10	Adit 3	15.9	IV	T-4	15.3	2.01
11	AA-10	195.54	III	T-1	17.54	2.00
12	AA-11	160	IV	T-5	33.49	2.69
13	AA-8R	112.5	III	T-2	30.27	0.00
14	Pipe gallery	22	III	T 3	2.2	0.00
15	VT	310	IV	T-4	38.64	0.00
16	BBC	81	III	T-2	25.09	0.00
17	D/S Surge Shaft Chamber 3	17.3	IV	T-4	33.86	0.00
18	DSSS-3	EL:580	III	OP-2	36.04	0.00

19	D/S Surge Shaft UP Chamber 4	15	IV	T-6	40.72	0.00
20	DSSS-4	EL:611	III	OP-8	36.76	9.32
21	AA-6 (Link Adit)	6	IV	T-4	31.64	0.00
22	TRT-3 D/s	500	III	T-1	25.27	0.00
23	TRT-3 U/s	146	VA	T-2	4.5	0.00
24	TRT-4 D/s	612	VA	T-3	18.18	1.99
25	TRT-4 U/s	110	VA	T-3	4.5	0.00
26	TRT Out Fall	70.00	IV	SM-3	34.98	1.38
27	Vertical Penstock-5	EL: 704.167		M-3	9.6	0.0
28	Vertical Penstock-7	EL: 704.355		M-3	8.9	0.00

Table 5

Maximum displacement observed by MPBX

SNo.	Component	Maximum sensor displacement w.r.t. initial @ sensor depth , RD		
		Crown	**U/S wall**	**D/S wall**
1	U/S surge shaft Chamber-3	1.51mm @ 10m, RD 16.6m Cable Damaged Since Nov'18	-6.80mm @ 15m, RD 16.6m, EL: 873m	11.01 mm @ 5m, RD 16.6m,EL 862m
2	U/s Surge Shaft-3	**Gallery Side EL: 857m**	**Adit Side EL: 856m**	**Adit Side, R/s EL: 856m D/s Wall**
		-5.00mm @ 2m	3.55mm @ 10m	4.65mm @ 2m
		Adit Side , L/s EL: 856.00m, U/s Wall		
		-0.98mm@5m Cable damaged since April'19		
3	U/S surge shaft Chamber-4	2.65 **mm** @ 2m, RD 21m	2.21**mm** @ 2m, RD 21m, EL: 873m	**-4.13mm** @ 2m, RD 21m
4	U/s Surge Shaft-4	**Adit Side Wall, EL: 857m**	**Gallery Side Wall EL: 856m**	**Adit L/s EL: 855m, U/s Wall**
		3.94mm @ 10m	4.56mm @ 10m	**2.04mm @ 5m**
		Adit Side Wall, EL: 790m	**U/s Wall EL: 790m**	**D/s Wall, EL: 790m**
		-2.79mm @2m	-1.60mm @ 10m	-0.74mm @2m
5	BVC	-22.66mm @ 2m, RD 72.13m	-7.57mm @ 15m, RD 35.605m, El:722m	19.93mm @ 15 m, RD 35.60 m, El:722m
6	PAC	-4.96mm @ 10m, RD 80.69m	-5.62mm @10m, RD 6.18 m	-9.66mm @ 10m, RD 6.18m
7	PH	-0.60mm* @ 5m, RD 87m	0.79mm @ 20m, RD 12m	5.32 mm @ 5m, RD 37m
8	PH-UDG	**-5.59mm** @ **10**m, RD 41m		
9	D/S surge shaft Chamber-3	-1.96 mm @ 10m, RD 13.8m	-2.44mm @ 5m, RD 13.8m	16.72mm @ 2m, RD 13.8m, EL 644.50m
		Adit Side EL: 613.393	**Link Tunnel Wall EL: 613.393m**	
		1.58mm @ 5m	0.51mm@m	

10	D/s Surge Shaft-3	**Link Tunnel Side Wall EL: 580M**	**D/s Wall EL: 580M**	**U/s Wall EL: 580M**
		4.21mm @ 15m	-1.13mm @15m	-2.53mm @ 15m
11	D/S surge shaft Chamber.4	**1.77mm** @ 10m, RD 13.8m Cable Damaged since July'17	**-3.42mm** @ 15m, RD 13.8m EL 649 m	**-6.78mm** @ 5m, RD 13.8m, EL: 649.50 m Cable Damaged since Jan'18
12	D/S surge shaft-4	**U/S wall EL:613.6m**	**EL: 613.711m End wall**	**EL: 613.650m D/S wall**
		-0.12mm @ 10m	1.77mm @ 5m	0.48mm @5m Cable Damaged since March'20
		Link Tunnel wall EL:613.603 m,		
		-2.00mm @15m		
13	Link Tunnel	7.16mm @ 15m RD: 21.50m	1.37mm @ 10m RD: 21.50m	-9.25mm @ 2m RD: 21.50m

Note: (-ve reading) Internal Convergence dominates, (+ve reading) Wall Convergence Dominates

Table 6
Rock bolt load increment w.r.t initial installed load

Component	**Maximum rock bolt load increments w.r.t. initial installed load (Tons)**		
	Crown	**U/S wall**	**D/S wall**
U/S SURGE SHAFT-3		1.10 Tons @ EL: 812.25	1.60Tons @ EL:812.25
U/S SURGE SHAFT- 4, EL: - 806.25		2.90 Tons @ EL:803.25	3.23 Tons @ EL:806.25m
		EL: 790.650m Gallery Side Wall	
		2.33 @ EL:790.65m	
BVC Load cell on Rock Bolt		3.07 Tons @ RD: 18.93	1.07 Tons @ RD: 23.42
BVC Load cell on Cable Anchor		47.65 Tons @ RD 74 m	67.99Tons @ RD 43 m
Rock Bolt Load Cell PH	9.55 Tons @ RD 138m	11.94 Tons @ RD 138m	9.33 Tons @ RD 38m
Cable Anchor Load Cell PH Gallery		41.13Tons @ RD 3.15 m	16.79 Tons @ RD 135.5m
D/S SURGE SHAFT-3, EL: 593.50m			3.40Tons @ EL: 593.50m
D/S SURGE SHAFT-4, EL:602.60m			9.90@ EL: 602.60m
Cable Anchor Load Cell Link Tunnel Between DSSS Chamber 3 & 4			3.37@ RD: 10m, EL: 645.5m
Cable Anchor Load Cell Out Let		5.10Tons @ RD -5 m EL: 620m	20.01Tons @ RD -50 m EL: 622m

Note:(-ve) reading indicates relaxation of load, (+ve) reading indicates accumulation of load

3. Conclusions:

The most important aspect of the interpretation part is the calculation of the expected value of the parameters being monitored. The recorded values of various parameters are being compared with the expected values to ensure collection of meaningful data. Another effective way to validate instrument reading is through routine visual observation. Observation of the monitored area can provide early warning signals, such as tension crack or evident seepage, which may not be picked up by nearby field monitoring instruments and can also guide remedial actions. The following conclusions can be drawn from analyzing the instrumentation and monitoring records till date.

Based on the instrumentation work carried out in Tehri Pumped Storage Plant, significant movements are being observed in BVC, PAC, Downstream Surge Shaft and Chambers. These movements may be due to adverse geology encountered during excavation. Considering the case of BVC and PAC which is very critical component of project wherein at some locations first warning limit has been breached. Special precaution has been taken during further excavation in this area by control blasting in various steps to avoid any mishap. Additional rock supports is also suggested by designers and implemented at site immediately. The monitoring data of other project components lies within warning limit and considered to be safe. Further, it has also been proposed that the monitoring shall be continued as per present frequency and if found any change in data in terms of increment immediately frequency of reading should be increase and counter measures to be taken without any delay for the safety of project component.

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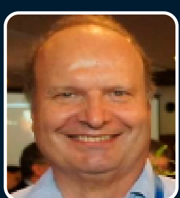

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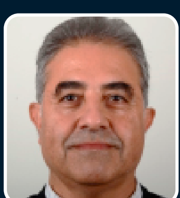

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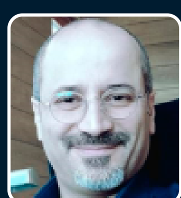

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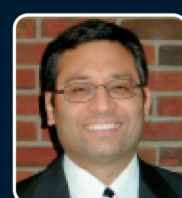

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